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THE ASSOCIATION was organized in Boston, Mass., on June 21, 1882, with the object of providing its members with means of social intercourse and for the exchange of knowledge pertaining to the construction and management of water works. From an original membership of only TWENTY-SEVEN, its growth has prospered until now it includes the names of 800 men. Its membership is divided into six classes, viz.:

A Member shall be an officer or employee of a public or private water works, an engineer, chemist or other person qualified to aid or interested in the advancement of knowledge relative to water works.

An Honorary Member shall be a person of acknowledged eminence in some branch of water supply or of engineering.

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An Associate shall be either a person, firm or corporation engaged in manufacturing or furnishing materials or supplies for the construction or maintenance of water works.

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The initiation fees and annual dues are as follows:

	Initiation Fees		*Annual Dues*
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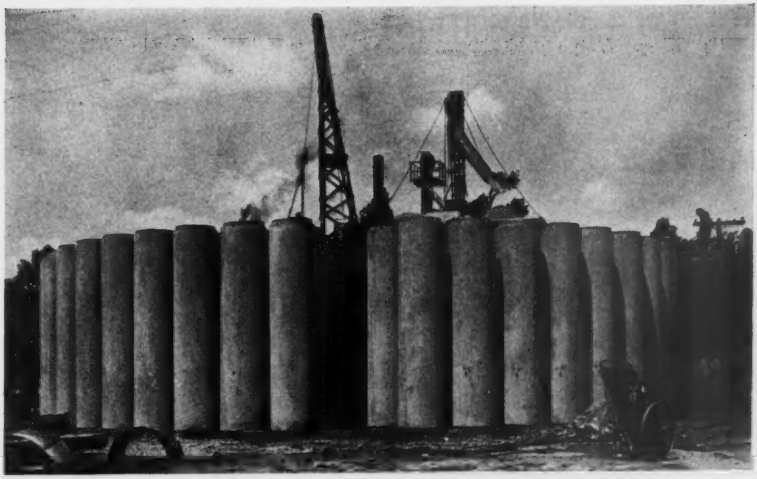
This Association has at least eight regular meetings each year, of which five are held in Boston, one in northern New England, one in southern New England, and one, the annual convention, held in September on such date as the Executive Committee may designate.



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The inherent weaknesses in any design may never be entirely eliminated, but even the most advanced design can be improved by applying the technical development of the future. It would insult the intelligence of engineers and scientists to claim that any meter made today will not be supplanted by a better design within the next twenty-five years. It would be difficult, indeed, to find anyone who thinks that the marvelous automobile of today will not be superseded by a better one tomorrow.

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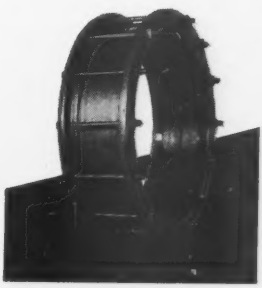

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New England Water Works Association
ORGANIZED 1882

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CONSTRUCTION OF THE DELAWARE AQUEDUCT

BY ROGER W. ARMSTRONG*

[*Read September 25, 1940.*]

The Delaware Water System and the events following its selection as a new source of water supply for New York City which delayed the start of construction for eight years were described briefly in a paper by the late Walter E. Spear published in the June 1937 number of this JOURNAL. Construction work on the project was started in February 1937 following the award of two contracts for portions of the Delaware Aqueduct. The work has progressed rapidly since then until at the present time (September 1940) the main portion of the aqueduct, that between the Rondout and Hill View reservoirs is approximately 50% completed. The main elements of the system and their relation to the existing supplies of the city are shown in Plate I.

Description of Delaware Water System. The project in brief includes the diversion of three Catskill mountain streams, the Rondout Creek, a tributary of the Hudson River, and the Neversink River and East Branch of the Delaware River, both located in the Delaware River basin. Storage reservoirs, of sufficient capacity to assure the diversion of a large proportion of its total runoff, will be constructed on each of these streams and will provide a total dependable yield of 540 m.g.d., 100 m.g.d. from Rondout Creek, 70 m.g.d. from the Neversink River, and 370 m.g.d. from the East Branch of the Delaware River. Water from the last two sources will be delivered to Rondout Reservoir through branches of the Delaware Aqueduct 6 and 26 miles long respectively. From Rondout Reservoir the total supply will be transported through the Delaware Aqueduct, a distance of 85 miles to Hill View Reservior, which is located in Yonkers just outside of the New York City line and is the main distributing reservoir for the Catskill system.

Description of Delaware Aqueduct. The Delaware Aqueduct,

*Deputy Chief Engineer, New York Board of Water Supply, New York, N. Y.

throughout its entire length, is a pressure tunnel located deep in the bed rock. A number of considerations influenced the adoption of this type of conduit. On the west side of the Hudson River, the terrain which the aqueduct must cross, consists of a succession of ridges and valleys, the trend of which is approximately normal to any reasonable aqueduct location. The valley bottoms are below and the tops of the hills, above the hydraulic gradient so that the possible amount of aqueduct in open cut at the hydraulic grade, which is the least expensive type, would have been relatively small. The aqueduct would have to be carried through the ridges in tunnel and, on the most favorable alignment which could be found, the greater portion of its length would be below the hydraulic gradient which would require a type of construction that would resist internal pressure. On the east side of the Hudson River, although conditions are favorable to the use of a larger proportion of open-cut construction, the region through which the aqueduct must pass is devoted to country estates and high-class residences. The expense of acquiring a right-of-way through most of this area would be high, and the disturbance caused by the actual construction work would be extremely objectionable, if not intolerable. In addition to these physical reasons for the selection of a pressure-tunnel aqueduct, the advantages of this type in construction, maintenance and operation were given due consideration. Since no surface rights need be acquired for the tunnel, the cost of right-of-way, except at the shaft sites, is nominal. The annoyance from construction operations to residents of the locality through which the aqueduct passes, is a minimum, existing only at the points of access which, for the Delaware tunnel, are from two to five miles apart. A pressure tunnel is more permanent than other types of construction and requires practically no maintenance. Since it always flows full and under pressure, it is sensitive to control, regulation of flow at the inlet end being immediately felt at the outlet. Finally it offers the maximum of protection against sabotage or accidental damage.

The horizontal alignment of the aqueduct, which with this type of construction might have been a straight line between the Rondout and Hill View reservoirs, deviates from such to a considerable extent in order to satisfy certain requirements of safety and flexibility in operation both of the Delaware system and the city's water supply as a whole. The location is such that the flow will pass through West

Branch Reservoir in the northerly portion of the Croton watershed, and provision is made for intercepting other high-level waters of the Croton System. Physical connection with the Catskill System is made at Kensico Reservoir, through which the Delaware water will flow, and at Hill View Reservoir, from which both the Delaware and Catskill supplies will be admitted to the two delivery tunnels that feed the city's distribution system. A slight deviation to the west from Kensico Reservoir outlet is made to reach a tract of land owned by the city and retained as a site for future purification works.

These location requirements effect a natural division of the aqueduct into three sections. The most northerly, 45 miles long and 13½ ft. in finished diameter, extends from Rondout Reservoir to West Branch Reservoir of the Croton system. The middle section extends from West Branch Reservoir to Kensico Reservoir of the Catskill system and is 23 miles long and 15 ft. in diameter. The southerly section, between Kensico and Hill View reservoirs, is 13 miles long and 19½ ft. in diameter. In addition, there are by-pass tunnels between the influent and effluent chambers of West Branch and Kensico reservoirs, each 2 miles long and 15 ft. in diameter.

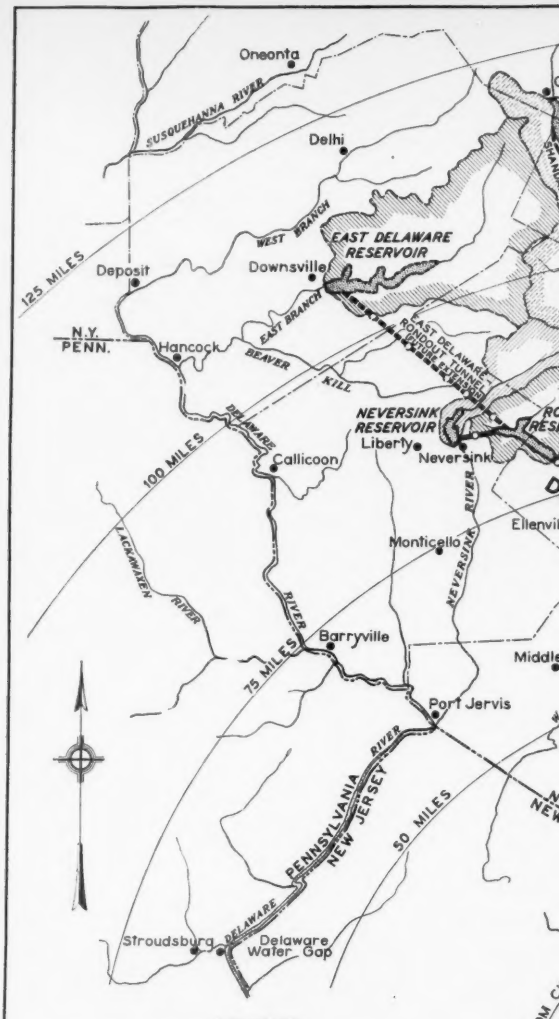
Beyond a minimum distance below the surface of sound rock required to resist internal pressures, the vertical alignment of the aqueduct, as shown on the profile of Plate I, was determined by geological conditions and the requirements for drainage. Each one of the three sections of aqueduct drains in both directions to a low point located in the vicinity of some natural channel at which facilities will be constructed for unwatering the tunnel. The effect of geology on the vertical alignment was to deepen the tunnel at several places, where zones of disintegrated or faulted rock were known to exist and where tunnelling at a higher elevation would have been difficult and hazardous. Three principal locations of this character occur at the Rondout Creek Valley, the Hudson River Valley and at Kensico Reservoir, but others of less importance at points along the line were indicated by geologic and sub-surface investigations.

The tunnel is being constructed through 32 shafts, all but five of which have been equipped for the transportation of men and materials during the construction period and the majority of which will eventually perform some function in the operation of the completed system. Three shafts, one in each of the three sections of tunnel, will be

TABLE 1—DATA RELATING TO THE SHAFTS OF THE DELAWARE AQUEDUCT

Shaft No.	Finished Diameter Feet	Depth Feet	Purpose	Contract	Cost
1	14	982.2	Surge shaft and breather.	340	$500,000.00*
2	14	825.0	Construction only.	332	
2A	14	1550.8	Surge shaft and breather.	332	$1,443,503.87
3	14	838.4	Construction only.	332	
4	14	877.0	Connection with Catskill Aqueduct.	333	
5	14	1010.0	Construction only.	333	
5A	14	1015.0	Connections for local supply.	333	$2,042,334.52
6	26½	680.0	Drainage for Rondout-West Branch tunnel. Connections for local supply.	333	
7	14	765.0	Construction only.	334	
8	14	949.0	Breather.	334	
9U	14	314.0	Waterway.	334	$1,784,732.36
9D	15	317.0	Waterway.	334	
10U	15	328.0	Waterway.	334	
10D	15	437.0	Waterway.	334	
11	16	313.0	Connection with Croton system. Connections for local supply.	322	Included in Table 2
12	14	359.0	Construction only.	322	
13	Elliptic**	350.0	Drainage for West Branch-Kensico tunnel. Connection with Croton system. Connections for local supply.	323	Included in Table 2
13A		314.0	Constructed on contractor's initiative and for his convenience only.	323	
14	14	560.0	Construction only.	323	
15	14	525.0	Construction only.	324	Included in Table 2
16	14	407.0	Construction only.	324	
17U	15	320.0	Waterway.	336	
17D	18	1020.0	Waterway.	336	
18U	18	1024.0	Waterway.	336	
18D	19.42	464.0	Waterway.	336	
19U	19.42	449.0	Waterway.	336	
19D	19.42	526.0	Waterway.	336	
20	16	431.5	Connections for local supply.	336	$3,947,542.32
21	16	495.3	Connections for local supply.	336	
22	16	367.4	Connections for local supply.	336	
23	Elliptic†	322.0	Drainage for Kensico-Hill View tunnel. Connections for local supply.	336	
1A-N	18	484.0	Waterway.	224	Included in construction of City Tunnel No. 2
			TOTAL		$9,718,113.07

*Very approximate. Included in contract for Merriman Dam.
**26 ft. 4 in. by 23 ft. 4 in.
†32 ft. 2 in by 22 ft.



LEGEND

The eighty-five mile Delaware Aqueduct will be driven from thirty-one shafts from 310 to 1580 feet deep. With its southern extension, City Tunnel No. 2, recently finished, it will make a continuous deep rock tunnel, 105 miles long, and from 13'-6" to 21'-0" diameter inside the concrete lining. The shafts and tunnel will be concrete lined throughout.

As shown in profile below, shafts in solid black to the top will carry water to or from the surface. Shafts marked 'A' will afford access to the tunnel. All other shafts are for construction only and will be sealed at top and bottom.

S - Denotes shaft that will act as surge shaft.

T - Denotes shafts that will have connections for the supply of neighboring communities outside of New York City.

C - Denotes shafts that will connect with reservoirs of the Croton System.

CA - Denotes shaft that will connect with the Catskill Aqueduct.

D - Denotes shafts that will contain pumps for emptying the tunnel when required.

Gate houses and valve chambers shown ⊕ and ● will contain the gates, valves, meters, etc. for controlling and measuring the flow of water into and out of the tunnel. Those marked ● will be underground.

Gate houses as marked will control the flow of water as follows:

N - from Neversink reservoir into the Neversink-Rondout tunnel.

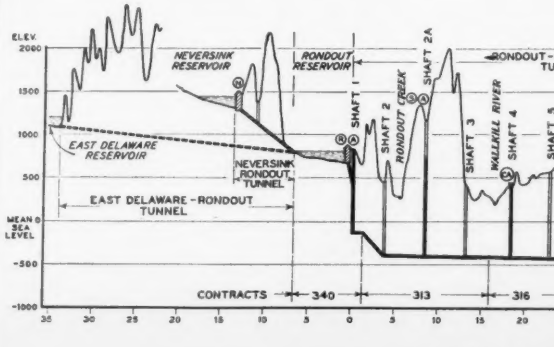
R - from Rondout reservoir into the Rondout-West Branch tunnel.

W - into and out of West Branch reservoir or through the West Branch By-pass tunnel.

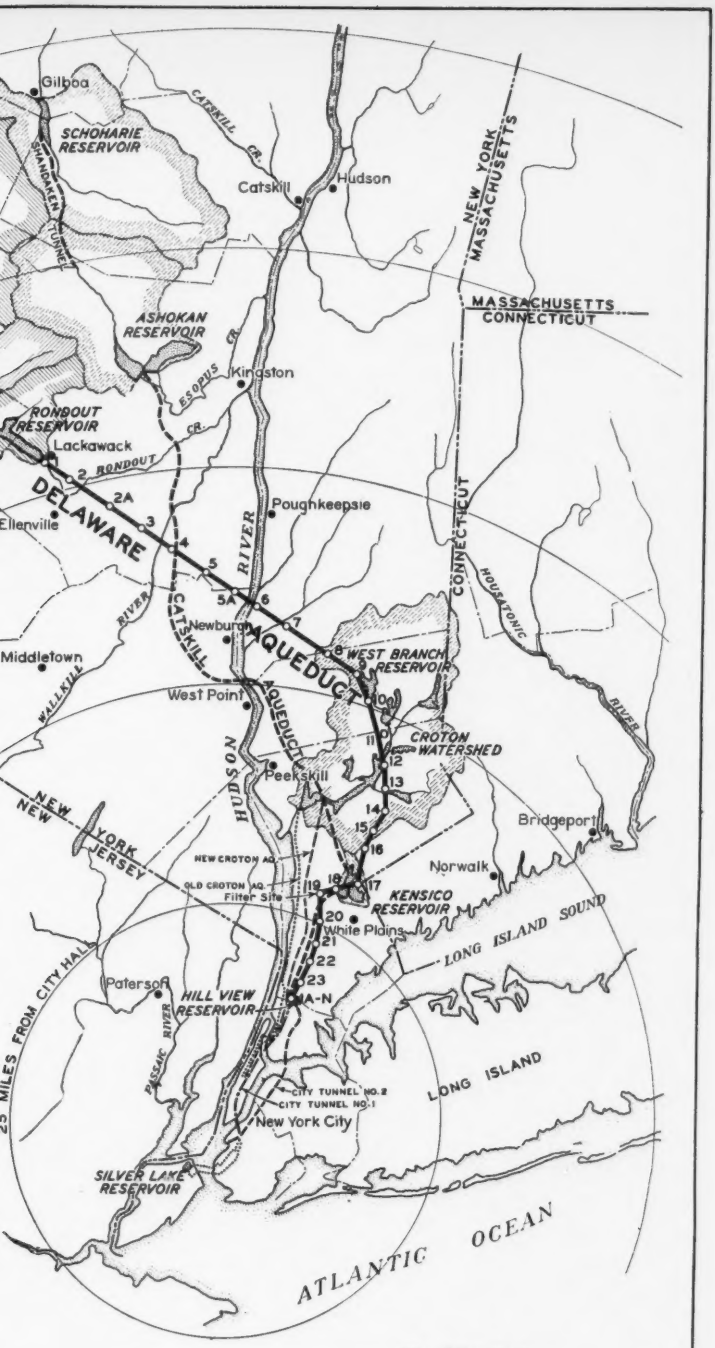
K - into and out of Kensico reservoir or through the Kensico By-pass tunnel.

F - to and from the Eastview filters when these are built on the site already owned by The City.

H - into and out of Hill View reservoir or through the Hill View By-pass tunnel



PROFILE OF



City of New York

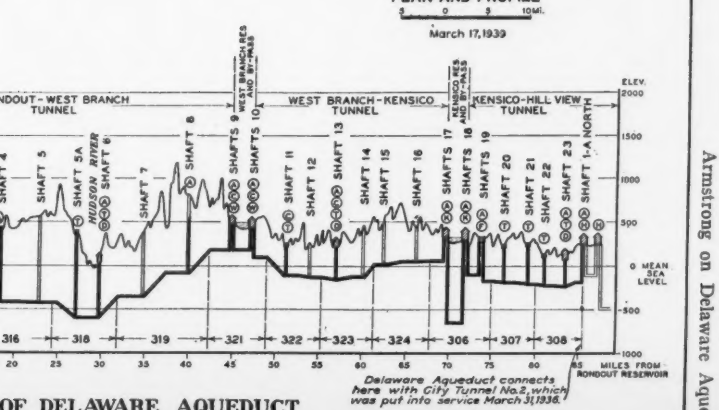
BOARD OF WATER SUPPLY

DELAWARE AQUEDUCT

PLAN AND PROFILE

5 0 5 10 Mi.

March 17, 1939



Delaware Aqueduct connects here with City Tunnel No. 2, which was put into service March 31, 1936.

OF DELAWARE AQUEDUCT

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PLATE I

Armstrong on Delaware Aqueduct

TABLE 2—DATA RELATING TO TUNNEL CONTRACTS FOR DELAWARE AQUEDUCT

Contract No.	Tunnel Length Feet	Tunnel Diameter Feet	Structures Other Than Tunnel Included in Contract	Bid Cost of Tunnel and Appurtenances
340	5,200†	13½	Merriman Dam, Rondout reservoir control works and appurtenances.	$1,300,000.00*
313	75,100	13½	None.	$18,916,650.00
316	48,100	13½	Chamber for connection with Catskill aqueduct.	$11,111,111.11
318	36,600	13½	Valve chamber at Shaft 5A and drainage chamber at Shaft 6.	$10,679,710.00
319	55,900	13½	None.	$11,431,975.00
321	11,900	13½	Influent and effluent chambers at	
	23,100	15	West Branch Reservoir.	$11,924,450.00
322	33,400	15	Shafts 11 and 12; valve chamber at Shaft 11.	$10,829,530.00
323	30,600	15	Shafts 13 and 14; drainage chamber at Shaft 13.	$10,810,166.50
324	37,100	15	Shafts 15 and 16.	$10,985,200.00
306	18,850	15	Influent chamber and coagulating	
	16,030	19½	plant at Shafts 17: Effluent chamber and aerator at Shafts 18; filter connection chamber at the top of Shafts 19.	$21,333,333.33
307	26,820	19½	Valve chambers at Shafts 20 and 21.	$10,097,135.00
308	29,140	19½	Valve chamber at Shaft 22; drainage chamber at Shaft 23; chamber superstructure at Shaft 1A North.	$11,971,885.00
			TOTAL FROM TABLE 1	$9,718,113.07
			APPROXIMATE TOTAL COST OF DELAWARE AQUEDUCT EXCLUSIVE OF EQUIPMENT.	$151,109,259.01

*Approximate estimate.

†Does not include about 3200 ft. of inclined tunnel between Shaft 1 and Rondout effluent chamber.

retained as drainage shafts and will be equipped with pumps and other apparatus necessary for the unwatering of the tunnel. In Shaft 4, located at the point where the alignments of the Catskill Aqueduct and Delaware Aqueduct intersect, gates and valves will be installed for

making a physical connection between the two systems. Eleven shafts will serve as waterways through which the water may pass to or from the surface, and eight shafts, including the three drainage shafts, will be equipped with risers and control gates for connections by which nearby communities may be supplied. Two shafts will have installations of valves and piping for connections with the Croton System, and three shafts will be left open with breather pipes at the top for the relief of surge. Nine shafts only are provided solely for construction purposes and will be sealed when the tunnel is completed.

With the exception of two of the drainage shafts, which were made elliptical in order better to accommodate the equipment they are to contain, all shafts are circular in cross section. Those which are for construction purposes only are 14 ft. in diameter within the finished lining. Others are larger, their size depending on their function in the operation of the aqueduct or on the equipment which they are to contain. Pertinent data relating to the shafts are shown in Table 1, and Table 2 contains information relating to the tunnel and the contracts under which it is constructed.

All tunnels are excavated to an approximately circular cross-section and subsequently lined with concrete, the inner surface of which is a true circle. The lining is provided to improve the hydraulic qualities of the aqueduct, to prevent deterioration and caving of the rock walls and roof, and to avert outward leakage when the aqueduct is in service. The latter function is especially important in the greater portion of the tunnel where the hydraulic gradient will lie well above the ground water level. Every effort is made during its construction to bond the lining as closely as possible to the rock against which it is placed. All rock débris is removed from the tunnel, and all dust or other coating is removed from the rock surfaces before concreting is started. After the concrete has been placed, unfilled voids, shrinkage spaces, cracks, and joints, as well as open seams in the rock behind the lining, are filled by pressure grouting through pipes left in the masonary or through holes drilled for that purpose. Ordinarily no reinforcement is used in the linings, since the tunnel is so located that the weight of the superimposed sound rock will resist the bursting pressure when the tunnel is in operation and the lining is designed of sufficient thickness to resist whatever pressures may come on it when the tunnel is empty. However, in zones where the rock has been

severely shattered or disintegrated, heavy reinforcement and steel interlinings are introduced into the concrete as insurance against rupture and excessive leakage resulting from compression of the surrounding material. Typical cross sections of the tunnel are shown in Fig. 1.

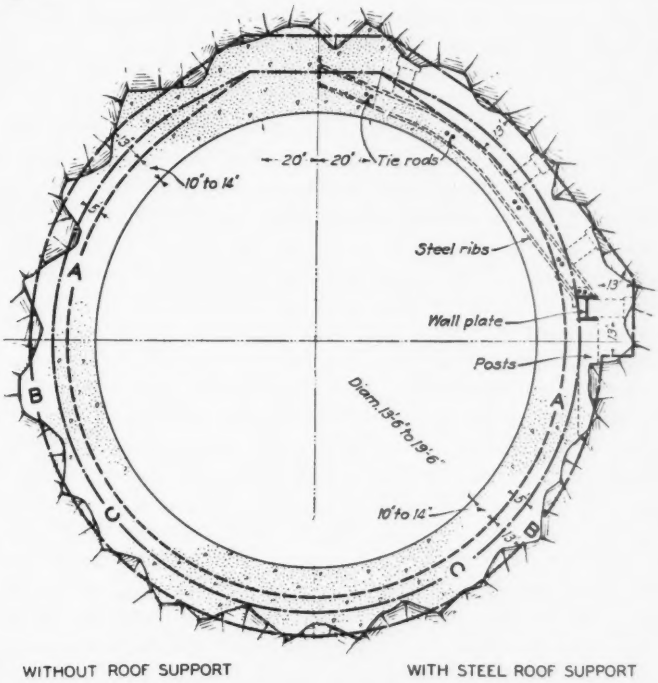


FIG. 1.—TYPICAL CROSS-SECTIONS OF DELAWARE TUNNEL.

The alignment of the aqueduct was not determined with sufficient finality to start construction until an extensive program of geologic reconnaissance and sub-surface investigation had been completed. In this work, the records of the geology of the region and experience with the several formations obtained during the construction of the Catskill Aqueduct were of great value as a guide. Where field reconnaissance showed no outcrops of rock, wash borings, followed

by diamond-drill borings, were made at frequent intervals to determine the rock profile along the selected line. Where abnormal conditions in the formation were found or were suspected, series of borings were made until the character of the material that would be encountered in tunnelling was determined in sufficient degree to permit prospective bidders to evaluate the difficulties. In this work, 647 borings were made aggregating 2,335 ft. in water, 41,092 ft. in earth, and 48,067 ft. in rock. These were made under 14 contracts at a total cost of $366,563.

Shaft Construction. At the time when authorization to proceed with the construction was obtained, the necessity of bending every effort toward an early completion was urgent, particularly for the portion of the aqueduct between Kensico and Hill View reservoirs, in order to effect a safer and more efficient operation of the Catskill Supply. Therefore, the shafts of the tunnel between Kensico and Hill View reservoirs and the deeper shafts of the remaining portion of the aqueduct were let in separate contracts so that their construction might be in progress while the contracts for the tunnel work were being prepared. Four shaft contracts were awarded covering all but nine of the thirty-two shafts. Of the remaining nine shafts, the southerly terminal shaft, 1A North, had already been completed in connection with the construction of City Tunnel No. 2 and Shaft 1 and the six intermediate shafts on the tunnel between West Branch and Kensico reservoirs could be included with other work without delaying the completion of the work as a whole. The ninth shaft, Shaft 13A, was not originally contemplated but was introduced by the contractor and constructed at his own expense in order to facilitate the work of his contract.

These shaft contracts included the turning of the tunnel headings and the driving of short stretches of tunnel away from the foot of each shaft so that subsequent contractors for construction of the tunnel could immediately install their permanent equipment and proceed with tunnel excavation without delay. They included also several provisions that would ordinarily be a part of the contractor's account, such as enlargements for plant at the foot of the shafts, chambers for booster pumps in the deeper shafts and service pipes in the shaft linings. All of these provisions were for the purpose of speeding up the work.

Although rock was comparatively near the surface at all of

the shaft sites, a small amount of earth cover was required to be removed in most cases. Except at Shafts 18, this was accomplished by ordinary methods of earth excavation and timber sheeting and bracing, or steel-sheet piling, and in several cases by concrete curb walls. The two shafts at Kensico effluent, which are located on the shore of the reservoir where the rock surface was from 38 to 56 ft. below the surface, were excavated to rock by means of reinforced-concrete caissons intended to be sunk by open dredging but so designed that they could readily be equipped with decks and air locks if found necessary. The use of air, however, was not required.

Shafts in rock were excavated with derricks for the first 100 ft., after which the erection of head frames was required. A preliminary lining, which in many instances was also the final lining, was placed in each shaft as excavation progressed. No greater depth than 100 ft. of unlined shaft was permitted at any time, but the lining increments were generally much less than 100 ft., varying with the character of the rock, the presence of water, or the contractor's established schedule of operations. The lining of the shafts in this manner obviated the use of all temporary support for the shaft excavations and, by providing bulkheads for grouting, was of service in cutting off inflow of water.

If water in objectionable quantity was encountered during the drilling of a shaft round, an attempt was made to cut off the inflow immediately by grouting. Holes in addition to those required for excavation were drilled to intercept the water-bearing seams, and gated pipes were driven into them. Grout was then pumped into the hole at pressures sufficient to force the grout into the rock against the ground water head. Where the rock was too seamy to hold the grout or too weak to withstand the grouting pressure, a concrete blanket was placed in the bottom of the shaft, grout pipes being embedded in the concrete and connected with the drill holes in the rock.

When excavation was to be suspended in order to place a section of concrete lining, the muck from the last blasting round was not removed, but the top was levelled off to support the first lift of forms. The concrete was lowered in buckets to a platform on top of the forms and shovelled into place. Grout pipes were placed in the lining at joints and open seams, and wherever seepage occurred. If the accumulated seepage became objectionable before excavation was completed, the lining was grouted as soon as it was strong enough to with-

stand the grouting pressure; otherwise the grouting of the lining was done upon completion of the shaft.

Construction of Tunnel. Except for about one mile included in the contract for the Merriman Dam, the construction of the entire 85 miles of tunnel is included in 11 contracts. Two of these are for tunnel construction only, but each of the others includes also the construction of some operating appurtenances of the aqueduct, the most important of which are the shafts that were not included in the original four shaft contracts, the inlet and outlet control works at West Branch Reservoir under Contract 321 and the inlet and oulet control works at Kensico Reservoir together with the coagulating plant, aerator and filter connection chamber under Contract 306. The work included in these eleven contracts is shown in Table 2.

Plant Installation. Tunnel construction on all the contracts is characterized by substantial installations of plant, preponderant use of mechanical equipment, and systematic organization of the work for rapid progress. In the fundamentals of plant layout, equipment, and methods of tunnel excavation, all contracts are similar, differing only in relatively minor items of plant and details of procedure. On all contracts, excavations are advanced by full-face headings; drilling is done from drill carriages or "jumbos," on which are permanently mounted all the equipment required for full heading drilling, including platforms for the workmen and for drill steel (Fig. 2); mechanical mucking is universal, the same type of machine being used on all contracts; steel cars on narrow-gage tracks are used everywhere for the transportation of muck; and, except on one contract where Diesel locomotives are used, electric traction, generally by storage-battery locomotives, is employed.

Probably the greatest variation in plant is in the equipment provided for hoisting the muck from tunnel level through the shafts to the surface. All head frames are of structural steel ruggedly built and from 65 to 150 ft. high (Fig. 3). The hoisting engines are usually located at ground level with the cables running over sheaves at the top of the head frames, but at several of the shafts the hoists are mounted on top of the head frames. On only two of the contracts are loaded muck cars brought to the surface. On all others, the muck is dumped from the cars into large skips at tunnel level, and the skips in turn dump automatically into bins at the shaft head.

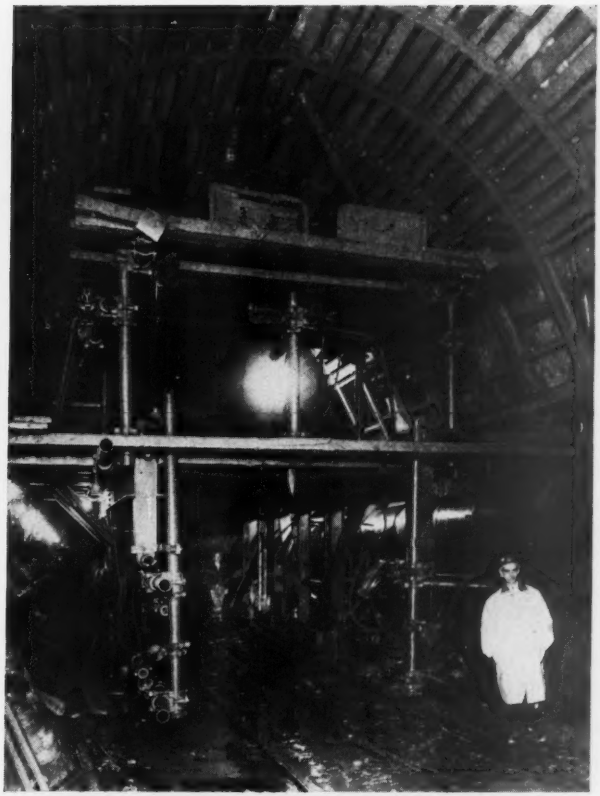


FIG. 2.—FRONT END OF DRILL CARRIAGE OR "JUMBO".

Excavation. The drilling patterns of the headings naturally vary with the size of the tunnel and the character of the rock. Ordinarily the cut holes are 12 to 14 ft. deep with the relievers and line holes 10 to 11 ft. deep. Forty-percent and sixty-percent dynamite is used in blasting, and advances run from 8.5 to 10 ft. per round. In the 19½-ft. tunnel, progress has been limited in most cases to 3 shots a day. In the 13½-ft. tunnel, however, it has been possible to obtain 6 shots per day and at times 38 to 40 advances in a six-day week.

FIG. 3.—HEADFRAME FOR TUNNEL CONSTRUCTION SHOWING MUCK BIN AND HOIST HOUSE.

For loading the muck into cars, all of the contractors use an electrically-operated machine, operating on a narrow-gage track (Fig. 4). It has at the front end a ½-cu. yd. bucket which is filled by crowding it into the muck pile by a forward movement of the whole machine, including an empty car that is coupled to the rear. The bucket then discharges its load by a backward throw onto a belt that deposits the load in the muck car. The passing of cars to and from

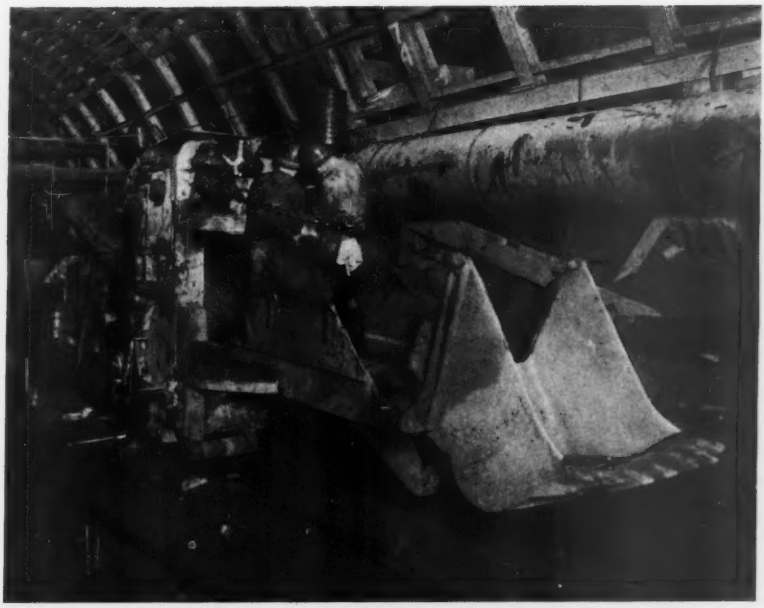


FIG. 4.—MUCKING MACHINE.

the mucking machine is generally accomplished by raising the empty from the rails and permitting the full car to pass under or to one side of it. The air—or electrically—operated hoist for raising the empty car, termed the "cherry picker" is usually located on the rear end of the "jumbo" (Fig. 2), which is moved back into the tunnel during the blasting and mucking operations. Some of the tunnels, particularly those of the largest diameter, are being driven through broken or disintegrated ground in which some form of support is necessary. This consists generally of steel "I" beams or "H" beams bent to the contour of the tunnel arch and supported on wall plates near the springing line (Figs. 5 and 6). Lagging, where necessary, consists of 3-in. "I" beams or light channels. Wooden blocking or wedges are placed between the steel ribs and the roof, but the use of timber is restricted to the minimum necessary for the safety of the

FIG. 5.—TUNNEL WITH STRUCTURAL STEEL ROOF SUPPORT.

support. Where the tunnel roof is not particularly heavy but is blocky or has a tendency to disintegrate on exposure to the air, pneumatically placed coatings of cement mortar (gunite) have been used with considerable success. When water in considerable quantities is encountered in drilling the headings, an attempt is made to cut it off by grouting before the heading is shot, the procedure being practically the same as that described for similar conditions in shaft excavation.

FIG. 6.—TUNNEL IN FAULTED AND DISINTEGRATED ROCK. EXCAVATION COMPLETELY SUPPORTED BY CONCRETE AND STEEL.

At several localities in the tunnel, zones have been encountered where severe faulting has occurred accompanied by considerable disintegration of the rock and where the ordinary methods of tunnelling were inadequate or unsafe and special methods had to be adopted. These zones carried water in troublesome quantities which was taken care of by one of two general methods. The first method consisted of forcing cement grout into an extensive system of drill holes with a view to filling the water-bearing channels as well as consolidating the rock; the second method permitted the water to enter the tunnel but confined it to small drifts or pipes low down on the sides of the excavation so that the rock might be drained and the tunnel advanced in fairly dry ground. Both of these methods were successful when applied to ground that was peculiarly adapted to their use. In most cases where such fault zones were penetrated, it became necessary to

abandon the full face headings and to proceed with the excavation by means of a series of small drifts in which the dangerous ground could be safely supported as the excavation progressed (Fig. 7). Typical operations performed in excavating tunnel are shown in Plate II.

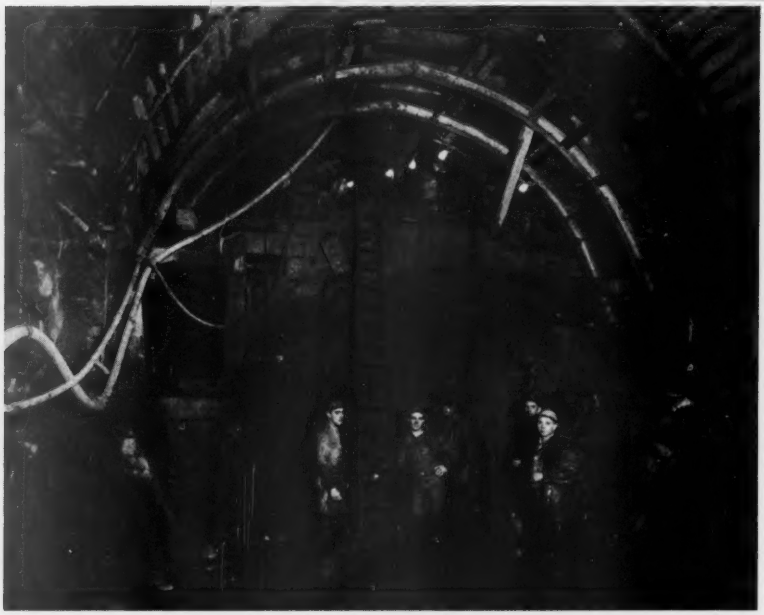


FIG. 7.—TUNNEL HEADING IN FAULTED AND DISINTEGRATED ROCK. EXCAVATION BY SIDE AND TOP DRIFTS.

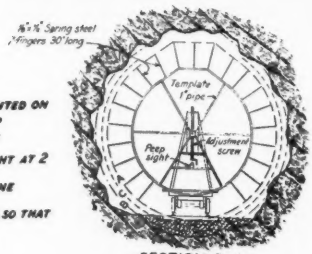
Construction of Tunnel Lining. During the tunnel-driving operations, a portion of the excavated material is allowed to remain in the invert to provide a level working floor and a foundation for the tracks over which the excavation equipment must operate (Fig. 8). The removal of this muck is the first step in preparation for placing the concrete lining. Considerable hand mucking is unavoidable in this operation, but drag lines and bulldozers have been found to be of material assistance. Before starting the placement of the final lining, it is the usual practice to construct concrete footings or curbs at the

TRIMMING

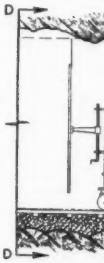
A TEMPLATE MADE OF 1" PIPE AND MOUNTED ON A FLAT CAR IS USED FOR MARKING "TIGHTS".

OPERATOR SIGHTS THROUGH A PEEP-SIGHT AT 2 LIGHTS, AHEAD, ON ℄ AND 3' BELOW SPRING LINE

TEMPLATE IS ARRANGED FOR ADJUSTING SO THAT IT MAY BE KEPT ON LINE AND GRADE.



SECTION D-D

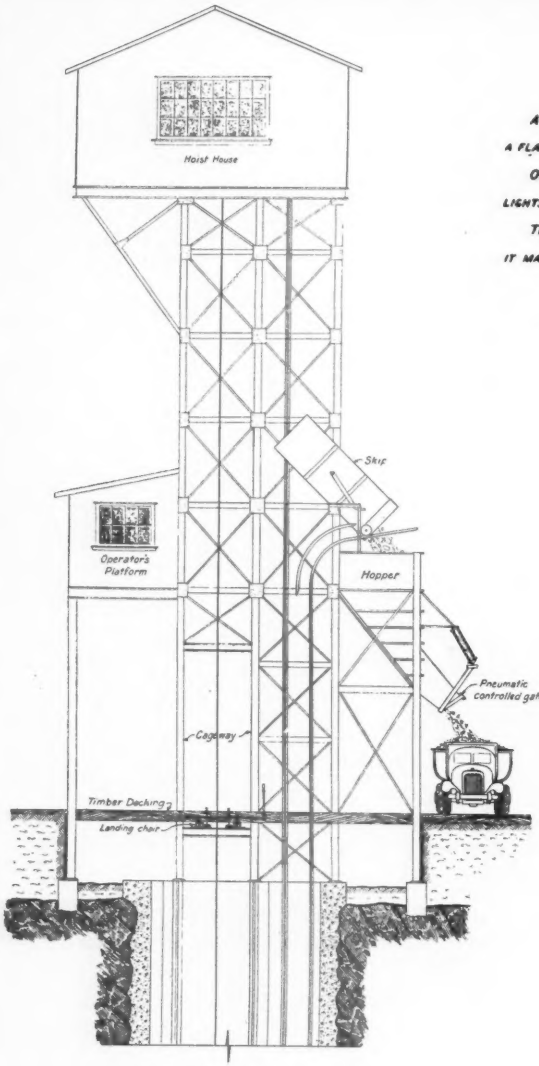


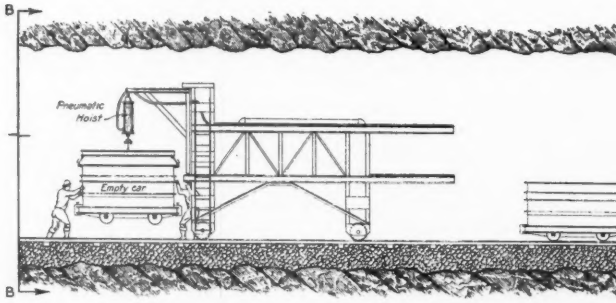
NOTE

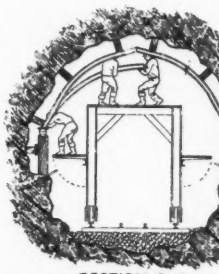
DURING THE MUCKING OPERATION THE DRILL CARRIAGE IS PLACED ABOUT 150 FEET FROM THE
A SWINGING BOOM MOUNTED ON THE REAR OF THE DRILL CARRIAGE, NEAR THE TOP, SERVES AS A
LAST EMPTY MUCK CAR AND SWING IT TO ONE SIDE CLEAR OF THE TRACK UNTIL THE ELECTRIC LOCO
CARS CLEAR WHEN THE EMPTY IS SWUNG BACK, LOWERED TO THE TRACK AND PUSHED UP TO THE MU

THIS OPERATION IS REPEATED UNTIL ALL CARS ARE LOADED THEN THEY ARE HAULED TO THE FOOT
CAR AT A TIME IS RUN INTO THE ROTARY DUMPER WHICH ROTATES ABOUT 180° TO DISCHARGE TH

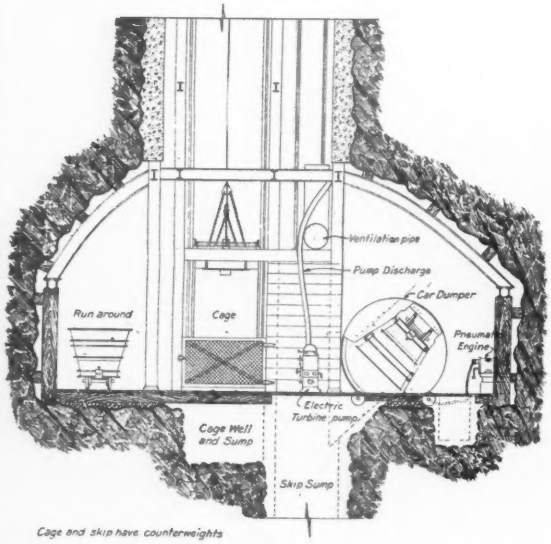
WITH TWO CAR LOADS, THE SKIP IS HOISTED TO THE SURFACE AND AUTOMATICALLY DUMPS INTO
MUCK IS CHUTED INTO TRUCKS AND HAULED AWAY FOR DISPOSAL.





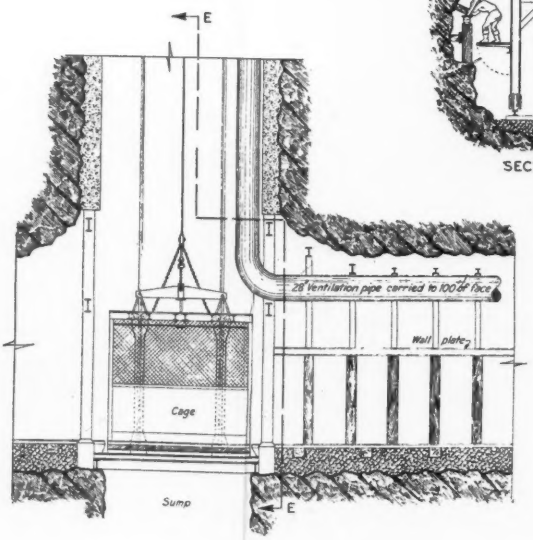


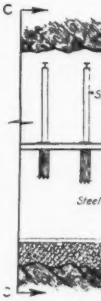
SECTION C-C



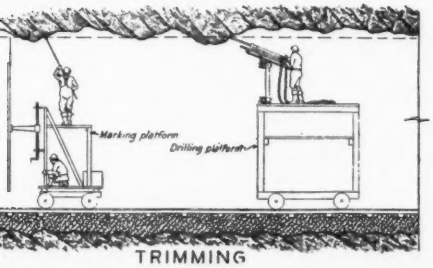
Cage and skip have counterweights

SECTION E-E

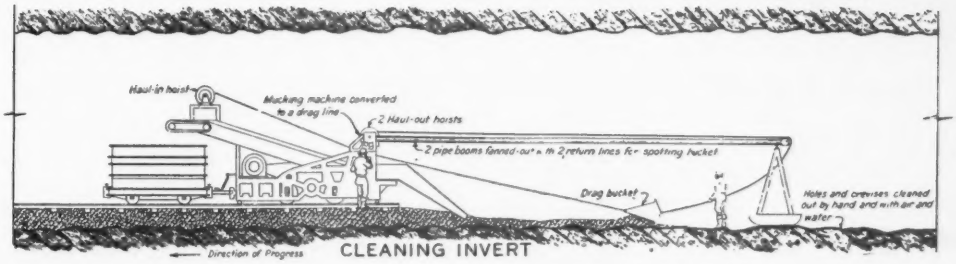




Drawn by E.M.C.

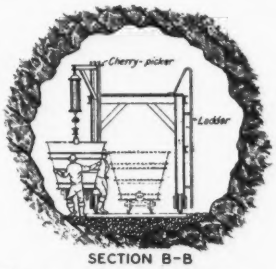


TRIMMING


Haul-in hoist
Mucking machine converted to a drag line
2 Haul-out hoists
2 pipe booms fanned-out with 2 return lines for spotting bucket
Drag bucket
Holes and crevices cleaned out by hand and with air and water
Direction of Progress

CLEANING INVERT

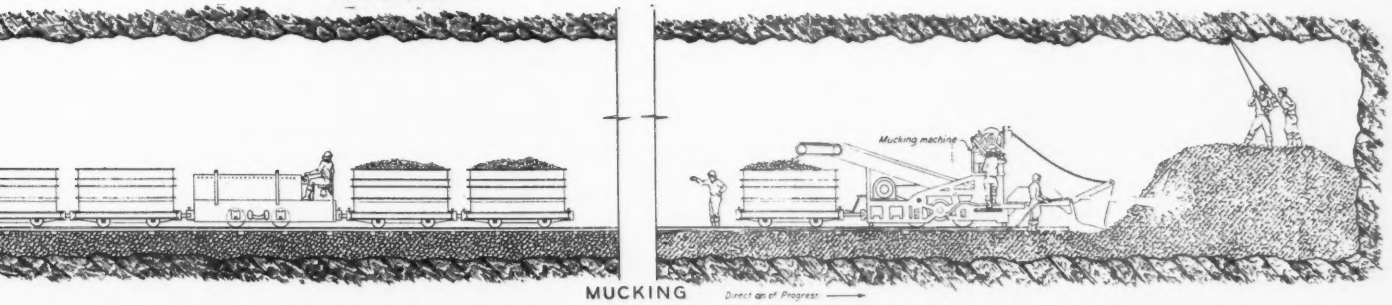
OM THE HEADING. AN AIR JACK ON
AS A "CHERRY-PICKER" TO LIFT THE
C LOCOMOTIVE PULLS THE LOADED
THE MUCKING MACHINE TO BE LOADED.
E FOOT OF THE SHAFT WHERE ONE
RGE THE MUCK INTO A SKIP
PS INTO A HOPPER FROM WHICH THE



SECTION B-B

MUCKING

AFTER THE HEADING HAS BEEN SHOT THE CONWAY MUCKING MACHINE IS MOVED UP TO THE MUCK PILE AND STARTS LOADING MUCK INTO CARS. THIS IS ACCOMPLISHED BY THE MACHINE FORCING THE DIGGING BUCKET INTO THE MUCK AND THEN RAISING THE DIPPER AND CHUTE BY HOIST CHAINS ALLOWING THE MUCK TO SLIDE BACK ONTO THE CONVEYOR BELT WHERE IT IS CARRIED TO THE REAR AND DUMPED INTO THE CAR



MUCKING

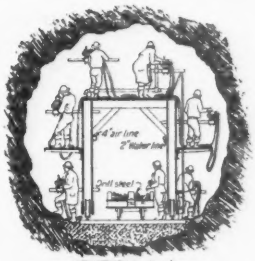
ROOF-SUPPORT

WHERE RUCK IS POOR STEEL ROOF-SUPPORT IS ERECTED BY THE MINING CREW AFTER EACH ADVANCE, FROM THE DRILL JUMBO, AND THE SUPPORT IS CARRIED WELL UP TO THE HEADING.

WHERE THE TUNNEL WAS UNSUPPORTED AT TIME OF EXCAVATION BUT LATER SHOWS SIGNS OF RAVELLING, DUE TO EXPOSURE, STEEL SUPPORT IS THEN ERECTED FROM A SPECIAL JUMBO.

GUNITE, A PROTECTIVE COATING OF CEMENT MORTAR, IS USED INSTEAD OF A LIGHT TYPE OF ROOF-SUPPORT IN THOSE AREAS, THAT MIGHT REQUIRE FREQUENT SCALING, WHERE THE ROCK ALTHOUGH SOUND HAS A TENDENCY TO CRUMBLE OR WEAKEN ON EXPOSURE.

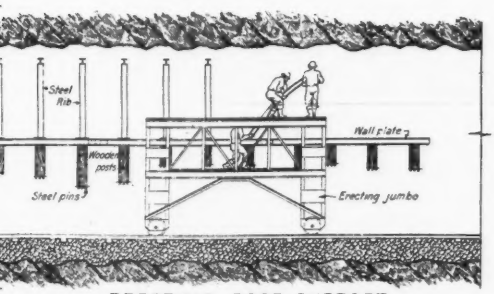
-C



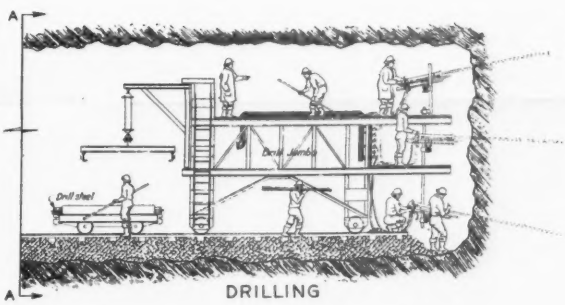
SECTION A-A

DRILLING

A FULL-FACED HEADING IS DRILLED WITH AUTOMATIC-FEED DRIFTER DRILLS MOUNTED ON A DRILL CARRIAGE OR "JUMBO". THIS CONSISTS OF A STEEL FRAME ABOUT 30 FT. LONG MOUNTED ON FLANGED WHEELS WHICH OPERATE ON STEEL RAILS LAID ABOUT 9 FEET APART. THE JUMBO HAS AN UPPER AND A LOWER PLATFORM AND IS FITTED WITH MOVABLE COLUMNS AND HORIZONTAL ARMS OF HEAVY STEEL PIPE ON WHICH THE DRILLS ARE MOUNTED



ERECTING ROOF-SUPPORT



DRILLING

City of New York
BOARD OF WATER SUPPLY
DELAWARE AQUEDUCT
EASTERN DEPARTMENT
TYPICAL TUNNEL DRIVING METHOD

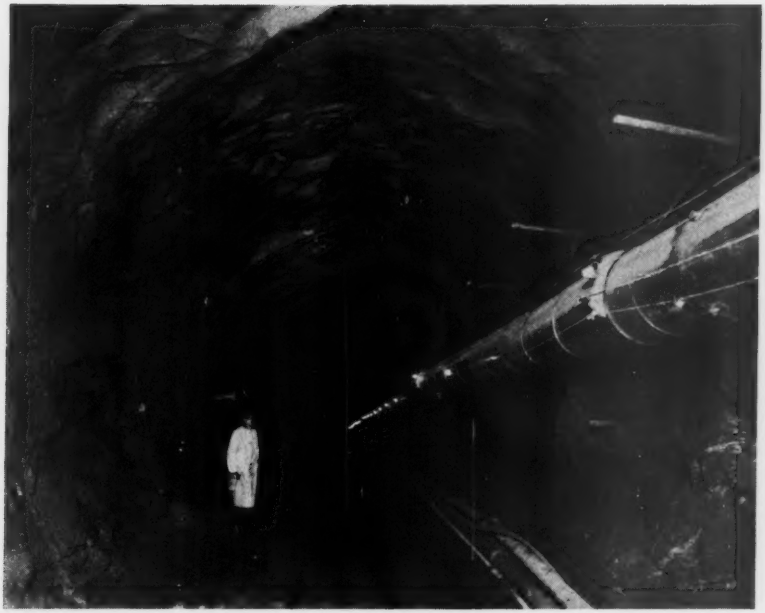


FIG. 8.—UNSUPPORTED TUNNEL.

sides of the tunnel. Steel rails set accurately to line and grade on top of these curbs serve as a guide to the setting of the invert forms and provide facilities for moving the forms and other concreting equipment (Fig. 9).

The concrete lining, the inner surface of which is a true circle, is placed in two operations. The invert section, which includes about 75 degrees of the circumference, is constructed first and, when sufficiently set, is provided with a track on which the carriage supporting the sidewall and arch forms and the cars and other concreting equipment may operate. The top surface of the invert lining is given its required shape by screeding and is finished by hand trowelling. The only forms required are straight bulkheads at the sides which form the radial joints between invert and sidewalls and also serve as guides for the screeds.

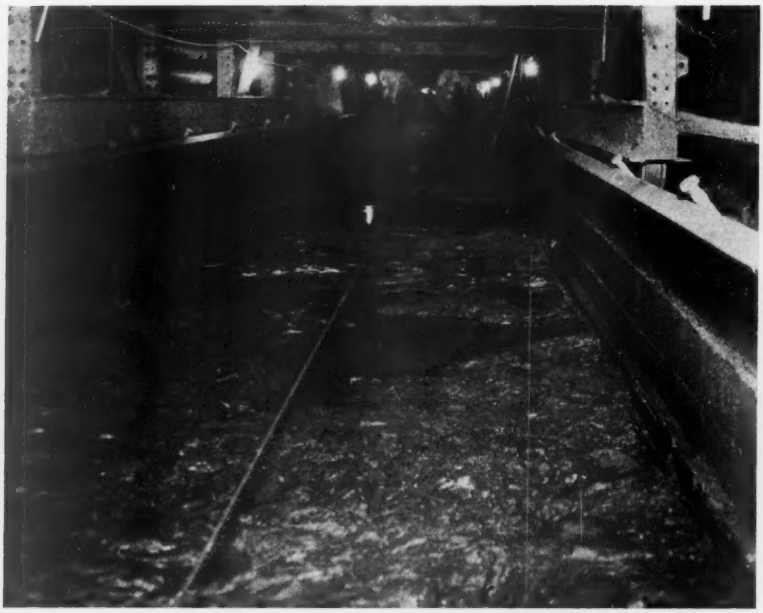


FIG. 9.—CONCRETING TUNNEL INVERT, SHOWING CONCRETE AND INVERT FORMS.

There are two exceptions to this general method of placing invert: one contractor uses a steel form to shape and finish the top surface and another has planned to place the invert as the last operation in the construction of the lining. The construction of the sidewalls and arch sometimes follows the invert construction closely, but the more common practice is to complete the invert in a considerable length of tunnel before starting the remainder of the lining. Sidewall and arch forms are fabricated of steel plates on steel ribs and when being moved are supported on a structural steel carriage that travels on the invert track (Fig. 10). When in position for concreting, the lower edges of the forms lap over the invert and are bolted to it using sockets set in the concrete for that purpose. The forms are hinged so that any section may be collapsed and made to pass through the section ahead. Removable plates provided at frequent intervals permit access to the back of the forms.

FIG. 10.—SIDEWALL AND ARCH FORMS.

The coarse aggregate for concrete may be either crushed stone or gravel, and the fine aggregate is required to be natural sand. The stone or gravel is used in two sizes, that which will pass a 1½-in. mesh and be retained on a ¾-in. mesh and that which will pass a ¾-in. mesh and be retained on a ¼-in. mesh. These sizes are mixed with the sand and cement in proportions designed to give a dense and impervious concrete having a cement factor of about 2 barrels to the cubic yard. Proportioning is done by weight at batching plants on the surface near the shafts. The mixing plants are located in some instances at the surface, in conjunction with the batching plants, and in others in the tunnel near the point of placement. In either case, the concrete is delivered to the tunnel level through pipes in the shafts. Dry batches dropped through these pipes fall directly into closed cars in which they are transported to the mixer where the water is added. Mixed concrete is transported to the forms in agitator cars.

The concrete is delivered to the forms either by use of pneumatic placers or blowers or by pumps and is consolidated by mechanical vibrators. On some of the contracts, the concrete operations are intermittent, a definite length of forms between bulkheads being filled each day; on others, the placing is continuous for five or six days, no bulkheads being used except at the point where work is stopped at the end of the week. Both methods have given equally satisfactory results. Plate III shows diagrammatically a method of placing concrete lining using pneumatic placers.

The following approximate figures of the quantities involved in the major items entering into the construction are given as a measure of the magnitude of the work.

Rock excavation in shafts and tunnel	5,700,000 cu. yd.
Structural steel support	95,000,000 lb.
Concrete and masonry in shafts and tunnel	2,400,000 cu. yd.
Portland cement	4,800,000 bbl.

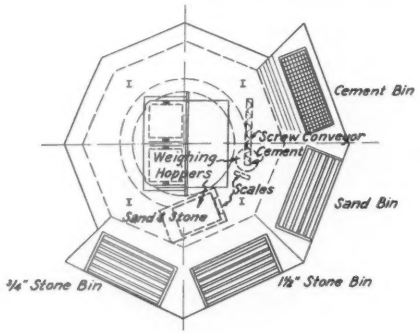
On account of the urgent need of additional water for New York City at the time at which construction of the Delaware Aqueduct was started, the contract requirements of progress were made as high as it was thought reasonable. It is satisfying to note that by following the construction methods that have been described none of the eleven contractors has failed to attain the required progress and many of them are months ahead of the schedule.

DISCUSSION

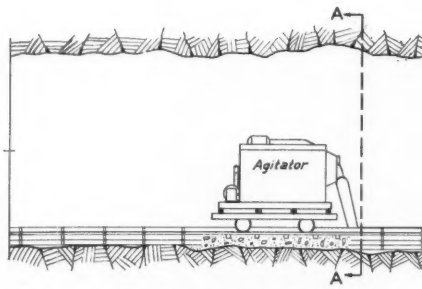
JAMES F. SANBORN.* As long ago as 1929, the Board of Water Supply of New York City made application to the State Water Power and Control Commission for approval of the development of the Delaware sources. In the injunction suit brought by the State of New Jersey, the Supreme Court decided in favor of the city on May 25, 1931. The city was at that time free to proceed in the development of the new source of supply, but it was January 28, 1937, nearly six years later, before a contract was let for shaft construction followed by the dam and tunnel contracts.

The Department of Water Supply, Gas, and Electricity operates

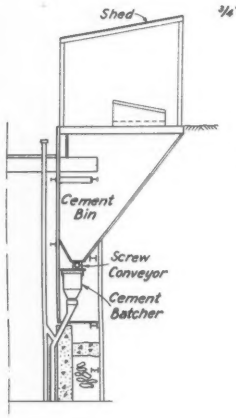
*Consulting Engineer, New York, N. Y.

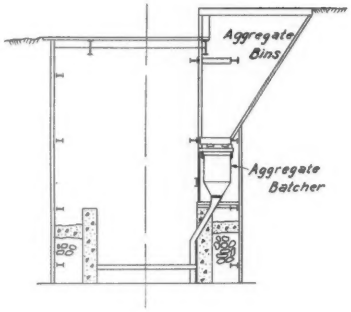


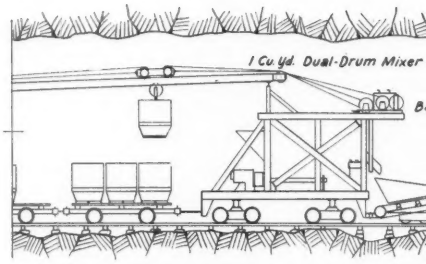
PLAN AT DECK

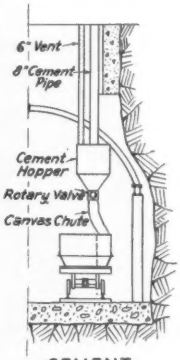


CONCRETING CURB-WALLS









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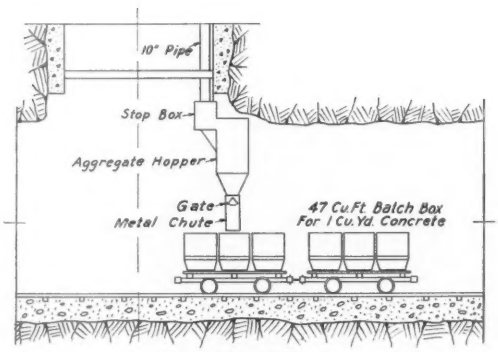

AGGREGATES

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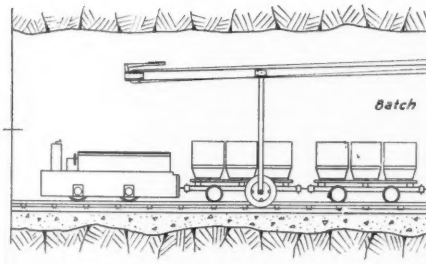
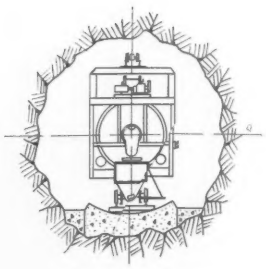
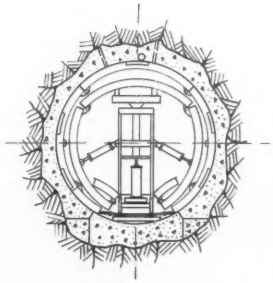




SECTION D-D



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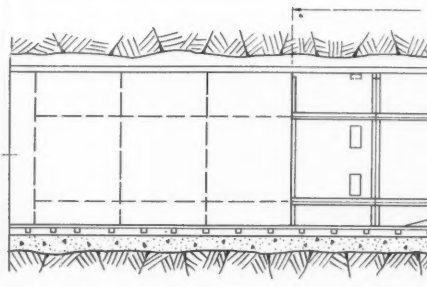

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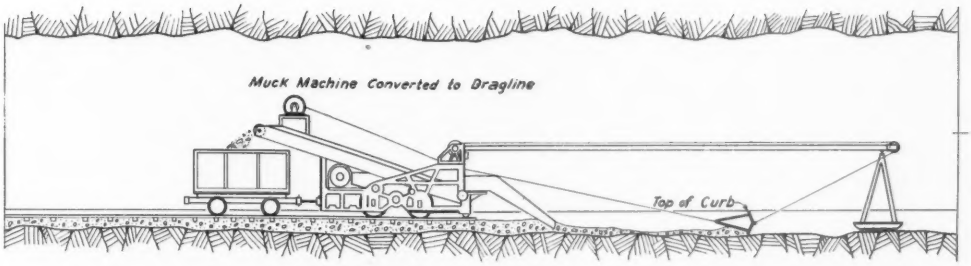
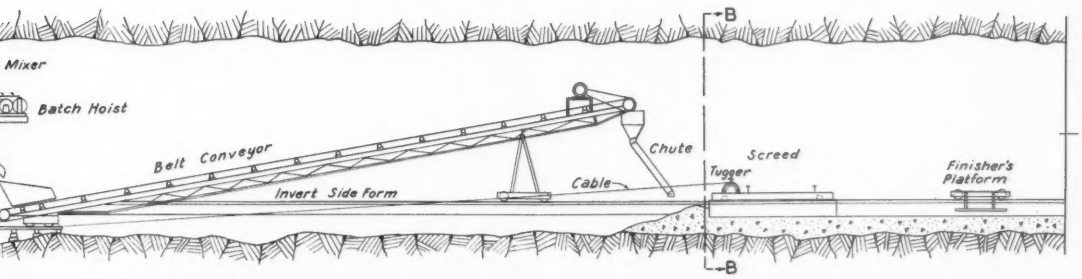
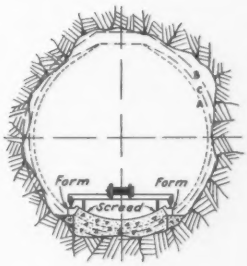


MUCKING INVERT

SECTION A-A



CONCRETING INVERT



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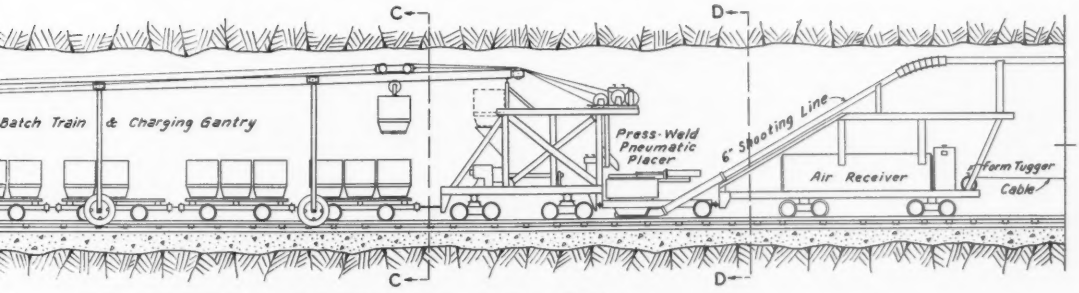
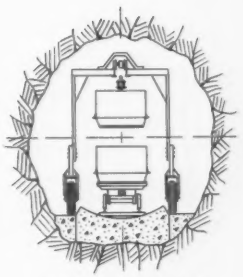


CONCRETING SIDEWALLS AND ARCH



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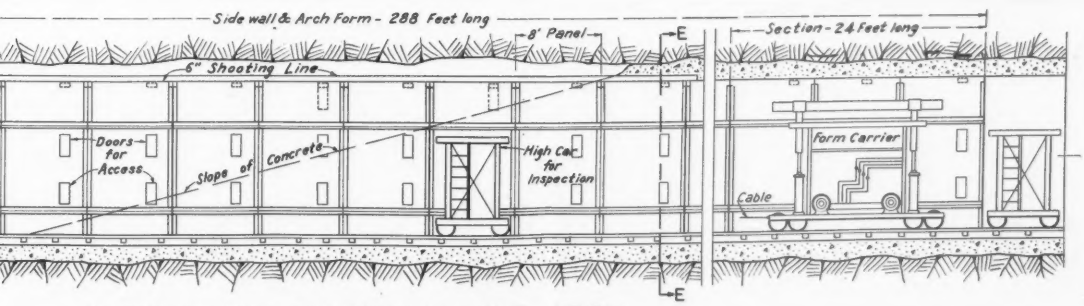

CONCRETING SIDEWALLS AND ARCH

City of New York
BOARD OF WATER SUPPLY
DELAWARE AQUEDUCT
CONTRACT 322
METHOD OF PLACING TUNNEL CONCRETE

the water supply and distribution to the several boroughs of the city. Outside communities, mainly in Westchester County, take an average of about 20 m.g.d. from the city aqueducts. During dry years and in hot weather, the daily use by such outsiders reaches 30 m.g.d. at times, a severe burden on the supply system.

Method of Operating System. In the distribution of water to various service areas, considerable pumping is necessary. The city operates the several supplies, namely the Catskill, Croton, Long Island and Richmond supplies, to produce the best results with the lowest costs. Since the Catskill Supply reaches the city at the highest elevation (El. 295), it is desirable to draw upon the main storage in Ashokan Reservoir (130 billion gal.) at the full capacity of the Catskill Aqueduct (625 m.g.d.) during as many days in the year as possible. This the city has been accustomed to do. Furthermore, the pumping from Long Island and Richmond wells is expensive, and for that reason such sources are used as little as possible, to save operating cost. Since pumping of Croton water is necessary (all except 130 m.g.d.), it is evident that the city can save money by using this source also as little as conditions require.

Demand. The average use of water from city sources during 1939 was 961.9 m.g.d. and from private water companies, 63.1 m.g.d., a total of 1025 mgd. for the year. Because of the drought during 1939, which continued until the month of March 1940, there was a heavy draft on the storage in the Ashokan and Croton watersheds, reducing the supplies to 40% of storage. Pumping from wells of the Ridgewood system was increased considerably during this period. Once before in recent years, during the drought of 1930-1931, the storage was reduced to 40% in the upstate reservoirs, leaving about 115 billion gal. in storage. The city bases its estimates of safe yield on the theory (1) that the storage in Kensico Reservoir (31.61 billion gal.) be drawn upon very little and (2) that the remaining storage (253 billion gal.) be never reduced more than about 75%, i.e. carrying not less than 60 billion gal. in upland storage as a minimum, or a total of 90 billion gal.

On this theory of operation, the estimated yield in the 100-year dry year is:

Catskill Supply	500 m.g.d.
Croton Supply	300 m.g.d.

Bronx and Byram Supply	10 m.g.d.
Total from upstate sources	810 m.g.d.
Long Island and Richmond supplies	140 m.g.d.
Total minimum supply	950 m.g.d.

In a discussion of the city's needs for water, up to the end of 1944 when delivery from the Rondout Supply is expected, the city assumes a safe yield for this short period of 850 m.g.d. from upstate sources and about 141 m.g.d. from wells on Long Island and Richmond (includes Layne Wells).

Anticipated Water Shortage. With a vigorous campaign of waste control, the city estimates the requirements in 1944 at 1029.2 m.g.d., but that the "dependable" supply in a dry year from available sources is 980 m.g.d. This does not count 11.5 m.g.d. from the Layne Wells in Queens County, operated at times by the city, which have never been approved by the State Water Power and Control Commission. The city expects that 100 m.g.d. from the Rondout will be available by the end of 1944, but that the safe supply will be some 50 m.g.d. less than the demand in that year.

City's Plans to Meet Possible Shortage of Water. In view of the great draft made on storage during the drought of 1939-1940, and the possibility that another dry time might occur before water can be delivered from the new Delaware and Rondout supplies, the city has made application to the Water Power and Control Commission for a supply from wells in Nassau County on the south side of Long Island, where the city has aqueducts of suitable capacity to carry the additional water estimated to be required before the end of 1945. After that date, it is assumed that the Delaware additions will satisfy all requirements. Once before, the city applied for a large supply from Long Island, which application was denied. Residents of Nassau County and other water users on Long Island have protested against the granting of the city's request, claiming that withdrawal of 50 m.g.d. would produce a serious reduction of the ground water storage, with danger of the back-set of salt water on the south side of the county, where the city already has a string of wells and infiltration galleries, in addition to wells of municipalities, water companies and private

developers of water. Such objectors have said that Nassau County has no water available for her future needs except from ground water sources in the county, while New York City has abundant upstate resources on which to draw, but for the development of which the city has not been willing to spend the money. The city argues that the proposed use of the new wells will be temporary, that after the Delaware water becomes available, such wells will be abandoned, that their use would in all probability be only seasonal and not continuous, and that if salt water were drawn in any well its use would be discontinued. But the objectors are still of the belief that the city would establish permanent control of the water, once permission were granted for its development. The case is now under hearings before the New York State Water Power and Control Commission.

KARL R. KENNISON.* I would like to describe some of our work which can be compared directly with the New York work, and some of the things that we are doing in a similar manner as well as some other things that we do a little bit differently.

Our Water Supply Commission in Boston has been operating since 1926 to extend the sources of water supply that were used by the district since the construction of the Wachusett Reservoir about 1900. The metropolitan district to which this water is supplied includes 20 cities and towns, the largest being Boston. In extending the sources of supply we constructed, first Quabbin Aqueduct, a tunnel 24.6 miles long to take the waters of the Ware River and the Swift River, and next Quabbin Reservoir, most interesting perhaps on account of its size. I believe it is the largest reservoir, solely for domestic water supply, in the world. This reservoir was formed by the construction of Winsor Dam and Quabbin Dike, which are two large earth dams similar in many respects to Merriman Dam of the New York water supply. This reservoir holds 415 billion gal., and the only excuse for its size is the peculiar and unique shape of the valley, which made it desirable and economical to build a reservoir much larger than necessary for the development of the immediate watershed and that of the Ware River, and one into which other watersheds can easily be diverted later.

The district consumes about 140 m.g.d., and the new source

*Chief Engineer, Metropolitan District Water Supply Commission, Boston, Mass.

increases the total safe yield to about two and three-quarters times that. This provides not only for increase in consumption by the present membership of the district, but also for enlargement of the district itself.

One comparison that can be made with the New York work is in the construction of our dams. Mr. Armstrong did not touch particularly on the construction of Merriman Dam, but, as you know, all of these dams were built by a similar method so far as the foundations are concerned, by sinking a row of caissons across the valley. This was done on account of the great depth to the sound ledge through glacial material of different degrees of porosity. For example, at Quabbin Dike it was necessary, in order to reach ledge, to go down about 130 ft. below the normal water level in the valley. At all the sites, this had to be lowered in order to enable the caissons to be sunk. When we started this method of construction, it was really new, although caisson foundations were used in the Bingham Dam in Maine. However, there the material through which they were sunk was just about 100% porous, there being no fine material in the ground at all. It was just a mass of large boulders, most of them as big as a man's head. It was simply a case of pushing the caissons down under very high air pressure, which could be done in the State of Maine at that time, there being no laws against it. For our work, we first picked out a fairly easy site and put down an experimental caisson. We experimented not only with the water level but also with the width and length of the caisson. The first one was maybe 32 ft. long, and 12 ft. wide. It was sunk at the Winsor Dam site where the depth was 102 ft. We experimented with different ways of providing pump inlets in the caisson itself. Then we would first sink and then pump. We also tried pumping while we were sinking, but it proved to be impractical to make any real progress in that way because the process of sinking, on account of leakage of air, seemed to separate the fine from the coarse materials and pack the fine materials in the intake. Then we moved over to the other site at Quabbin Dike and sank one caisson with the aid of an open well about 26 ft. square about 100 ft. to one side, first sinking from the well and then from the caisson, until we finally got down, using maximum air pressures of 48 lb. per sq. in. Finally at Winsor Dam we used a similar method and sank two caissons, not too close together, so that the pumping of one might not affect the pumping from

the other. That is the method that was used at Merriman Dam, and we feel quite sure that our experience, and the experience here, indicate that that is the best way: to sink two caissons without attempting to do too much other work until those caissons are down to ledge, and several pumping inlets are established in the sides of the caissons.

We found it best to construct a very substantial steel plate trough embedded in the permanent underpinning and to fill it with very carefully graded material, to be used as a permanent pumping inlet, one on the downstream side and the other on the upstream side. We had pumped for four months at Quabbin Dike before we let the contract, proving the feasibility of the method, and the water level was lowered about 41 ft. before the contract was let. Then the contractor lowered the level another 55 ft., so that it was lowered about 96 ft. altogether, and in general the air pressures were kept down to about 18 lb. per sq. in.

The dam embankments were built by the hydraulic fill method. We were very fortunate in having almost ideal material for this purpose since it did not have too many fines in it. I think at only one place was it necessary to waste fines. That is, the material was fine enough for imperviousness and yet not too fine for drainage, so that within a year after the completion of the dam the core was very firm at the bottom. One of the contractors at Quabbin Dike used trucks to convey materials from the borrow pits, but at Winsor Dam a great many feet of belt conveyor were used, some belts being as wide as 42 in.

I will now speak of some of the work that we have been doing recently, particularly the pressure tunnel work, so that I can compare it with some of the New York work.

The money appropriated for this job of extending the sources of supply was $65,000,000, and we saved out of that about $12,000,000; the Massachusetts Legislature two years ago decided to match that against Federal funds and construct as a PWA project a new pressure aqueduct which would take the water at the terminal of the old Wachusett aqueduct, about 278 ft. above sea level, and carry it into the district under pressure, paralleling and gradually superseding the old grade line aqueducts. Some of those aqueducts are very old: the Cochituate Aqueduct was built in 1848 and the Sudbury Aqueduct was built in 1878.

The principal interesting feature of our new pressure aqueduct, not directly comparable with any of the New York work, is the size of the large, cut-and-cover, precast, steel cylinder, reinforced concrete pipe. There is one section of two miles, 12½ ft. in diameter; then there is a section of tunnel about three miles long, followed by about 13 miles of 11½ ft. precast pipe. The pipe was all manufactured under one contract and was laid under five different contracts, very expeditiously. Within the next two or three years we are going to extend the pressure aqueduct into the heart of the district, by a deep pressure tunnel that will eliminate all of the ordinary high-service pumping at Chestnut Hill Reservoir.

In our tunnel construction, we have been very fortunate in the character of the rock encountered. Supports were required in only about 5% of the total length of the pressure tunnel. As to the method of lining, our contractor was very enthusiastic about it, and I have had a number of inquiries from other contractors about it. We accomplished the complete circular lining with one operation. There were two hinged sections on each side of the circular form which allowed the bottom to tip back and the two sides to swing in so that the whole form could be collapsed for telescoping; when put together it gave a complete circular section, with holes at the sides for access and inspection during the concreting operation.

As Mr. Armstrong has said, one of the principal headaches of the contractor, and the principal difficulty in this kind of work, is cleaning up the invert. The first tunnel that we built, Quabbin Aqueduct, was constructed as a grade-line tunnel, not as a pressure tunnel, and invert construction was entirely different. In pressure tunnel work, one needs to get the concrete into as intimate contact with the rock as possible. One of the things that retards progress is the need for satisfactory removal of the muck and all other materials on the invert, as the work proceeds. If some method could be devised to have, in addition to the apparatus for handling the form and bringing in the concrete, a conveyor of some kind to carry the muck back and get it out of the way more quickly, we would be able to proceed very much faster with the work. We were continually held back by the cleaning of the invert, but, in spite of that, the conractor was enthusiastic about this method.

The aggregates were mixed at the surface and dropped down the

shaft; the cement was dropped to a hopper at the bottom of the shaft, and then the aggregates and the cement were carried forward on the rails to the mixer at the forms. From that point the concrete was forced by air pressure into the top of the form. One of the difficulties of this method of lining is to get a good job in starting the pour at the invert. In spite of as careful vibration as we could provide, it was difficult to get all the air out of the bottom, and it was necessary to shoot that concrete over to one side in getting started, so that it would always be forced around in one direction, especially at the start of the pour. If some method could be devised for continuous pouring, such as Mr. Armstrong spoke of, so that concrete could be continuously poured for a whole week, it would be easier to get the air out of the invert because one would not have to pour up against a dead end bulkhead. At times, we had to do considerable patching of the surface on both sides near the invert.

LEONARD P. WOOD.* Mr. Charles M. Clark, Chief Engineer, Board of Water Supply, City of New York, was scheduled to discuss this paper, but late yesterday a situation developed which made it impossible for him to be here, and he has asked me to speak in his stead.

Mr. Armstrong has mentioned the length of the tunnel, 85 miles. It is a continuous tunnel 85 miles long, and it might be mentioned that in addition to that the southern extension of that tunnel, which will distribute the water to approximately 15 points in the city, is also a pressure tunnel 20 miles long, making a continuous line of pressure tunnel 105 miles long, and it is safe to say that that is far and away the longest continuous tunnel for any purpose ever built. It is probably about four times as long as any other tunnel of any size.

It is interesting to look at the changed conditions which have occurred since the previous tunnel work on the New York water system. It happens that tunnels have been built for the New York City water supply now for a hundred years. Just about a hundred years ago the old Croton aqueduct was built, involving a large number of small tunnels about 7 by 8 ft. in finished diameter, and at that time the longest one that they felt like undertaking was 1,263 ft. Most of the tunnels were very much shorter. The records of how that

*Designing Engineer, New York Board of Water Supply, New York, N. Y.

tunnel was built are lacking, but we can imagine that the methods must have been very crude.

About 50 years ago, the new Croton aqueduct was built, that aqueduct being about 30 miles long and almost all in tunnel, and a horseshoe section about 13½ by 14 ft. That 30 miles of tunnel was driven from 40 shafts and two inclines, so that you can see that at that time they were not prepared for very long headings between shafts. The shafts were from 400 to 7500 ft. apart, 7500 ft. between shafts being a pretty big engineering job at that time.

The best speed that could be made with the methods available at that time ran from 25 to 40 ft. a week, after they were well organized and in good ground. In places where the ground was not good they did not approach those figures.

About 30 years ago the Catskill aqueduct was built, and in favorable ground the best they did was from 55 to 70 ft. a week, 70 ft. a week being in a sound shale rock, easy drilling, good breaking, without water, and not requiring any support. That was the top, but there were many contractors who did not reach those speeds.

Twenty years ago the Shandaken tunnel, being a part of the second stage of the Catskill system, a tunnel of horseshoe design, about 11½ by 10 ft., one requiring considerable support, made the high speed of 100 ft. a week—an advance over the new Croton of three or four times.

The City Tunnel No. 2, the southern extension of the Delaware system, built about 10 years ago, in the hands of a very able and efficient contractor, working in much harder rock and a larger tunnel, 17 ft. in diameter, involving more bad ground, averaged, after it was under way, about 75 ft. a week, the lower speed being due to more unfavorable rock and a larger size of tunnel.

On the present work, the contractors have made quite extraordinary speed for work in the East. I might say that the magnitude of the job and the fact that important tunnel work in the West, particularly on the Los Angeles aqueduct, was closing up just about the time our work started, brought to us a corps of exceptionally able contractors with wide experience in rapid tunnel work. The result of that, and the result of the fine organization and equipment which they provided, was that the speeds of the tunnel excavation have exceeded even our best hopes. On the basis of previous experience the contract time schedules were set up on the assumption of a tunnel progress of from

350 to 400 ft. per month in tunnel excavation after the initial stage of the work close to the shafts and after getting organized.

In the old days it was pretty good performance if you got in one round per day, one shot per day. You would drill most of one shift and you would work two shifts to get the muck out by hand methods. On this work, the contractors from the start set up for themselves a schedule of three rounds per day, making the shot and getting the muck out of the headings on an eight-hour shift, and before the work was over, many of them were getting in more than three rounds per day. In the latter part of the work, on one contract, in favorable ground, the contractor got in six rounds per day every day of the six-day week for a period, and the average progress on this work, during the period of active work, after the work was well organized, on all 13½-ft. tunnel was about 200 ft. per week. That is including all delay due to bad ground. That is over a period of nine weeks, when some of the headings were making no progress at all due to bad ground. The average for the same nine-weeks period in the 15-ft. tunnel was 140 ft. per week, and in the 19½-ft. tunnel about 125 ft. per week, which far exceeds the contract rates. I might say that every tunnel contractor on tunnel excavation is ahead of the contract requirements, and many of them are far ahead.

A characteristic of the work which Mr. Armstrong has indicated is the elaborate plant which the contractors have provided. The desire for high speed and the high cost of labor on our work under the statutory requirement for prevailing wages—as it has worked out the labor costs range from $5 to $20 a day—have necessarily required the contractors to use mechanical equipment to the utmost, and without exception they have put in very complete and elaborate equipment, such as Mr. Armstrong has indicated, at a cost per shaft, before they really got started on the main job, of from a half million to three-quarters of a million dollars initial investment.

A characteristic of the work, aside from its magnitude and speed, is the care that has been taken to minimize accidents and industrial diseases among the personnel. Each contractor is required to have a full-time safety engineer, to have complete and adequate first-aid equipment at each shaft, with constant trained attendants, facilities for getting a doctor on the job quickly, and hospitalization arrangements, and all of them have carried out those requirements very well. In

addition to that, the Board of Water Supply maintains three full-time safety engineers who exercise general supervision over all the work, endeavoring to anticipate the possible sources of accidents, and to see that provision is made before the accident occurs to prevent it.

In addition to that, this work is perhaps to be called a pioneer in the matter of keeping the personnel free from silicosis, which is the lung disease caused by inhaling the dust of silica rock. Under the requirements of the New York State Labor Law, made comparatively recently, anybody doing rock drilling must use equipment approved by the State Department of Labor for the purpose of minimizing danger to operators from inhalation of dust. It so happens that our work started just at the time when the Department of Labor's investigations and program were maturing, and we got the benefit of these developments by the Department of Labor of the coöperation of the drill manufacturers in altering their drills, and of the contractors in adopting the methods, so that this job is probably far and away the best, so far as the protection of the men from the hazard of silicosis due to inhaling rock dust is concerned. The state Labor Law sets up a maximum number of dust particles to the cubic foot in the air, in the breathing zone of the men.

The dust is controlled in two principal ways. By the copious use of water and by copious ventilation. The drills which have been developed—called the New York State pattern of drill—differs from ordinary water drills in injecting a much larger quantity of water through the hollow drill steel, and a much smaller quantity of air. The result is that when dust is created by the impact of the drill, the dust is mixed with the water, and the amount of air blown through the steel for clearing the drill is so reduced as to minimize the spraying of that suspension of dust and water into the air at the face of the tunnel. The drill is so made that the operator cannot, through design or carelessness, start drilling until the full flow of water is going through the steel, so that to start the drill without any water is impossible with the present drill.

In addition to that, the State Code requires a minimum volume of ventilation for each drill. For the 3½ and 4 inch drifters used in these tunnels, 1200 c.f.m. are required. The contractors put in very unusual ventilating plants. You may have noticed in the pictures the very large ventilating pipes shown in each of the tunnel pictures. Those ventilat-

ing pipes run from 26 to 30 in. in diameter. There is a separate blower for each heading and a separate pipe from the top of the shaft. The blowers are from 100 to 150 horsepower, and the quantity of air delivered to each heading ranges from about 10,000 to about 14,000 cubic feet per minute.

In addition to the copious ventilation, and carrying the vent pipe as close as practicable to the face, boosters, or face blowers are used in some cases to get better circulation ahead of the end of the ventilating pipe, right up to the very face. Other contractors have used air filters on the drilling jumbo to clean the air, so as to have clean and filtered air at the working place. The results have been highly satisfactory. Whereas when we began the work of drilling we got something like 10,000,000 particles per cubic foot in the air, I think it is almost uniformly the case that the test shows considerably less than that amount now.

Another thing in which we find satisfaction is the sanitation of the working places, and the sanitary protection of the personnel and of the neighboring communities.

SANITATION OF CONSTRUCTION AREAS ON THE DELAWARE AQUEDUCT

BY FRANK A. MARSTON*

[*Read September 25, 1940.*]

The sanitary work herein described has been carried on not only to safeguard the health of men engaged on the construction of New York's new water supply and to protect people living on areas adjacent to the work, but to prevent the pollution of water in existing reservoirs that are a part of the water supply for nearly eight million people. Such objectives justify the exercise of more than usual care and the expenditure of substantial amounts of money in the building and operating of the works involved.

The first stage of construction of works for the additional water supply of the City of New York involves the building of a reservoir on Rondout Creek and the Delaware Aqueduct, an 85-mile conduit, from Rondout Reservoir at Lackawack to Hill View Reservoir just outside the city limits. This conduit is of the pressure type located at considerable depths below the surface in sound rock. The new work includes 30 shafts[1] along the 85 miles of aqueduct line. Eight of these are for construction use only and will thereafter be sealed, two will serve as surge chambers, and the others will become permanent parts of the control and water distribution works. For purposes of control and to tie in with existing reservoirs, the water will be brought to surface chambers at five points. The plan and profile shown on Plate I, which accompanies the paper by Roger W. Armstrong in this issue of the JOURNAL, gives the location of aqueduct shafts and Rondout Reservoir.

The building of these shafts together with Rondout Reservoir requires the carrying on of construction operations on the surface at 25 points. Nine of these construction areas drain to tributaries of the Hudson River. While none of these streams serves directly for public water supply, some are in localities used for fishing, bathing

*Consulting Engineer, Metcalf and Eddy, Boston, Mass.

[1]Shaft 1A-N was built with City Tunnel No. 2 and is not included in the works herein discussed.

and camping. Rondout Creek, one of these tributaries, is employed by industries to develop power and for process water. Eleven construction areas are located within the watersheds or on the shores of existing reservoirs of New York's present water supply, and one drains to a stream that feeds into the water supply of the city of Yonkers. Four other areas drain to tributaries of Bronx River, which flows for some distance through a popular parked area in Westchester County.

The general use of automobiles for transportation has enabled workmen to live at some distances from the jobs, with the result that there are no contractor's camps at any of the construction sites and only one-fourth of the sites have lunch rooms within the construction areas. The number of workmen required at each construction site has varied with the character of the work being done. In general, from 150 to 200 men are employed at each site, but at times the number has been increased to 300 or over. Construction work is carried on in 3 shifts, 24 hours per day, 6 days per week.

There has been a heavy use of water at each site, and a correspondingly large amount of sewage. Shower baths are provided for the engineers and for the contractors' forces, in addition to the usual lavatories, urinals and closets. Boot-washing sprays add to the water consumption, and waste through plumbing fixtures has been, at times, a material item of water waste.

Contractors' Water Supplies. The construction specifications require the contractors to furnish ample supplies of water of proper quality, with pipe distribution systems, elevated tanks, pumps and other appurtenances to give adequate water for fire protection and construction purposes, as well as for drinking and washing. All water, except that from approved public supplies, piped into the buildings or into the tunnels, which may be used for drinking, must be chlorinated. Water for construction purposes can be taken from sources separate from the drinking-water supplies.

For the six shaft areas nearest the city of New York, the contractors have piped water from local public supplies. In two other cases, water from a public supply has been transported by trucks to storage tanks at the work. At five shaft areas the water has been taken directly from nearby reservoirs of the New York system; at one other, the water has been pumped from the stream that is a

reservoir outlet; and at another shaft area, the water has been pumped from the Catskill Aqueduct. A shallow dug well at one construction site has been the source of supply, and at the other nine sites, wells from 128 ft. to 388 ft. in depth have been used. During extreme dry weather, a few of the deep wells have been so reduced in capacity that limitations on the use of water have been enforced. In addition to the main or official sources of supply, surface springs, wells and leaks in the tunnels have been used to supply drinking water, although this practice has not been generally encouraged.

In most cases, sterilization of the water used for drinking has been accomplished by treating it with a solution of sodium hypochlorite fed automatically through machines controlled by pump or meter.

Water for cooling air compressors and for use in the blacksmith shops has been taken, where possible, from nearby reservoirs or streams and returned thereto without treatment. Where the water had to be purchased or the supply was limited, a few water-cooling towers have been installed and the water re-used.

Following an investigation of the character of drinking water in pails and other containers as distributed to workmen on the surface and in the tunnels, the contractors were required to provide closed containers that would protect the water from contamination, thus ruling out the time-honored pail and tin dipper well-flavored with *B. coli* and oatmeal.

The quality of these water supplies has been checked at regular intervals through routine sampling and bacteriological analyses. Chlorine residuals have usually been checked daily.

Contractor's Plant. At each construction site there is a group of buildings, that at Shaft 9 on West Branch reservoir near the town of Carmel, shown in Fig. 1, being more or less typical. The engineers' offices, contractor's offices and the change house, have both water and sewer connections, and many of the other buildings have water connections. A number of hose connections are also provided for use in case of fires. At Shaft 9, the contractor's plant covers an area of about 7 acres.

Sewage Disposal. During preliminary construction operations and for the use of small groups of workmen located at a distance from centers of activity, the contractors have been required to provide

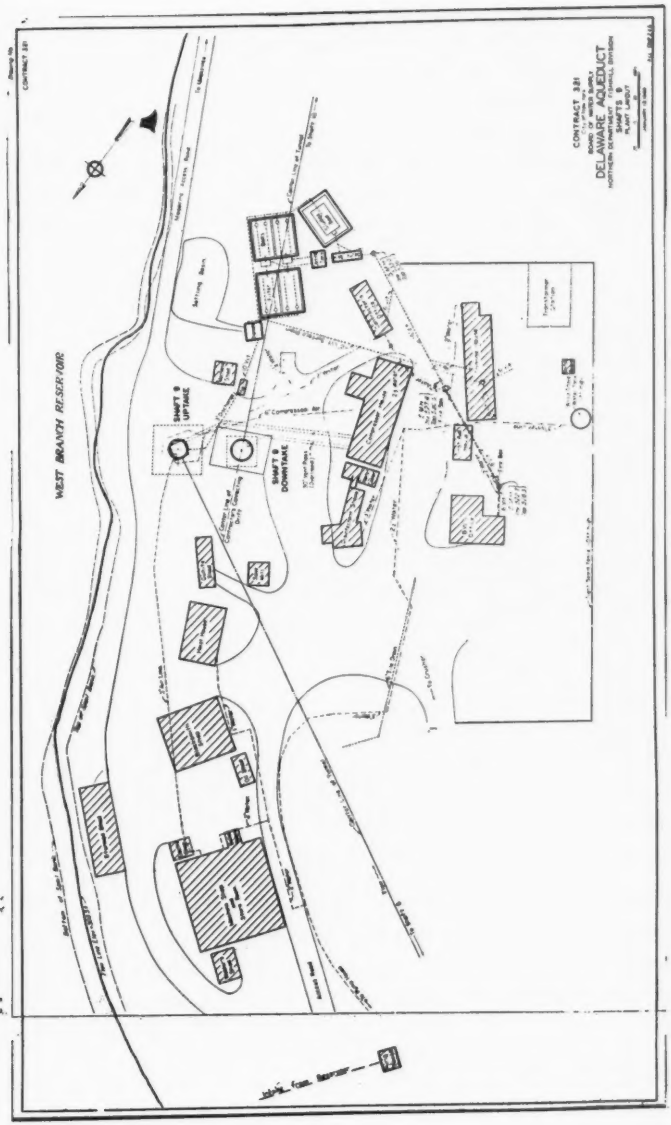

FIG. 1.—SHAFT 9. CONSTRUCTION BUILDINGS NEAR CARMEL, N. Y.

chemical closets with watertight removable receptacles. Similar arrangements have been made in the tunnels. Sodium hydroxide, chloride of lime, or similar disinfectants have been used to control odors and flies, and material from the closets has been hauled away and buried in approved locations or dumped into septic tanks.

Buildings having sanitary facilities have been equipped with plumbing fixtures of the water-flushed type, draining to existing public sanitary sewers in five construction areas, or to sewage-treatment plants in the other twenty. At all but two sites, the sewage-treatment plants consist of septic tanks, dosing tanks, intermittent sand filters and effluent chlorinators. The two exceptions are without filters, and only one of these has a chlorinator. Small open sludge-drying beds are provided at all but four plants, and in those cases the intent is to haul sludge away to some suitable point of disposal.

The sewage from these construction areas is quite weak because of the large amounts of wash water from the shower baths and lavatories, and the grease content is low. The quantities of sewage produced per capita have varied considerably but, in general, have been rather high, because of the extensive use of the washing facilities. It has been remarked that the workmen leaving these jobs are cleaner and better dressed than generally observed on large construction projects. Weekly average sewage flows during the first 18 weeks of 1940, for 13 of the construction areas showed variations from 8 to 54 g.p.d. per man, with a general average of 25 g.p.d.

Outlines of typical sewage-treatment plant structures are shown in Fig. 2. The exterior of typical plants, those at Shafts 19 and 2, are given in Figs. 3 and 4, respectively. The photograph of the former was taken before the filter housing was built. All of the septic tanks except three have a water capacity of 5000 gal. (24 hours' flow at the average rate). The three tanks are larger. Each septic tank is divided into two compartments by a central transverse wall so that the sewage flows through the first compartment and over a weir into the second, thence over a weir into a collecting chamber. Baffles deflect the flow from passing straight through the tank, and partitions at the weirs retain the scum. The tank bottom is steeply inclined to permit drawing sludge from the first compartment into the second and from either compartment to the sludge-drying bed.

The overflow from the septic tank passes through collection and

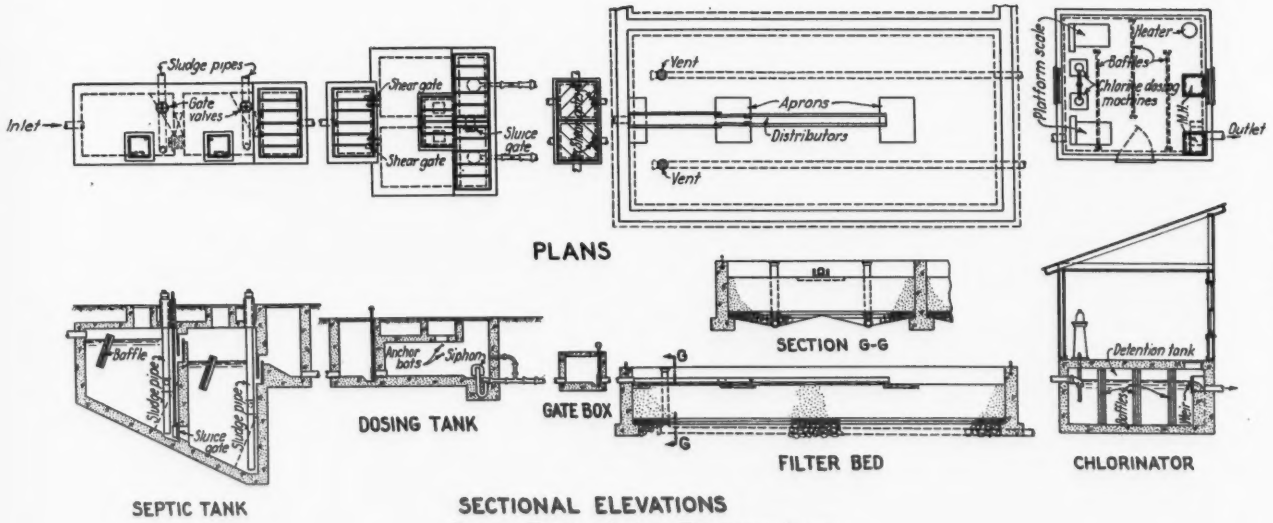

FIG. 2.—TYPICAL SEWAGE TREATMENT PLANT.

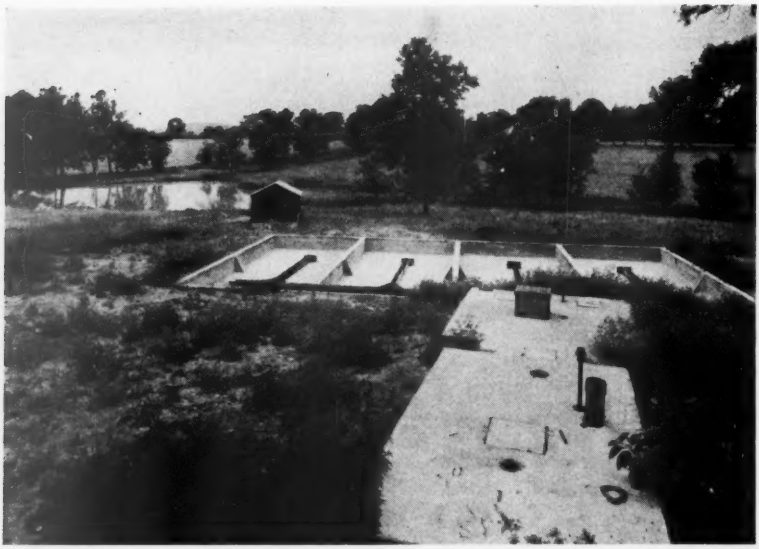

FIG. 3.—SHAFT 19. SEWAGE TREATMENT PLANT AND DRAINAGE CHLORINATOR. SAND FILTERS BEFORE HOUSE WAS BUILT. COCHRAN POND IN BACKGROUND.

distribution manholes to a two-compartment dosing-tank equipped with automatic siphons. The siphons are designed to discharge in rotation so as to put a dose of sewage, 3 in. deep, first on one bed and then on another, in accordance with the distributing gates being opened. As a rule, both compartments of the dosing tank are operated together as one tank.

In each plant there are four filter beds, each of about 510 sq. ft. area of sand surface, except in the three larger plants, one of which, the largest, has filters with sand areas of 910 sq. ft. per bed. Each filter bed contains 3 ft. of sand on layers of graded crushed stone that surround the lines of vitrified tile underdrains. Sewage is distributed over the surface of the sand by a wooden trough with adjustable outlets. The underdrains collect the filtered sewage, or effluent, and carry it to the chlorinator where a baffled contact basin is provided to mix the chlorine with the sewage and allow time for the chlorine to be effective in destroying bacteria. The specifications required that the

FIG. 4.—SHAFT 2. TREATMENT PLANT LOOKING TOWARD VERNOOY CREEK.

filter sand should be washed sand, passing an 8-mesh sieve, with effective size between 0.25 and 0.35 mm.

The septic tanks, manholes and dosing tanks are covered with concrete slabs or with wooden planks. The filter beds are covered with wooden houses having ample windows for light and ventilation, and the chlorinator basins are covered by substantial wooden houses to protect the chlorine equipment. The chlorinator houses are kept warm, and heaters have been provided, generally of an electric type, to raise the temperature of tray water during cold weather. The chlorinator basins were designed to provide at least 15 minutes of detention capacity for the average rate of flow.

Sludge-drying beds consist of a 6-in. sand bed in an open earth basin of an area equivalent to about 1.3 sq. ft. or more per capita. Thus far the sludge-drying beds have been used but little.

The filter beds were designed to treat 100,000 gal. of sewage per

acre per day, or 2.3 g.p.d. per sq. ft. of sand surface. On the design basis of 25 g.p.d. per capita, the population loading would be 4000 persons per acre. While the unit rate of sewage flow has been greatly exceeded at times, due to infiltration of ground water, waste and other causes, the population loading has generally not exceeded that expected.

At one plant for a period of 3 months, April 1 to June 30, the filters operated at an average rate of 5.2 g.p.d. per sq. ft., which is about double the rate for which they were designed. For this three-month period the average sewage flow was equivalent to 57 g.p.d. per capita, and the population load on the filters was equivalent to 4700 persons per acre. For one month, June, the rate of filtration was 7.7 g.p.d. per sq. ft. For this same month, the average sewage flow was equivalent to about 70 g.p.d. per capita and the population load on the filters was equivalent to 5625 persons per acre. It is not surprising that following such a period of overloading with water the old sand became clogged and had to be replaced with new. It should be remembered that the sewage is relatively weak and that there is very little grease, also that the sewage is quite warm. Prior to the three-month period that has been referred to, the filters had been receiving, for several months, an increasing load beyond that for which they were designed. At the time the sand in the filters was replaced, the filters had been in operation for about 2 years. During this period the surface of the sand had been scraped 3 times, removing 3 to 4 inches of sand each time. The effluent from the plant, even under these extreme conditions, has been excellent.

In general, where overloading of the filters with water has occurred, it has been possible to keep the filters in satisfactory operation by frequent raking of the surface and occasional removal of surface sand. In spite of the housing over the sand filters, there has been some freezing of sewage on the filter surface at several of the plants, but the warm sewage from the next dose has usually melted the ice and made it possible for the filter to keep in operation.

The New York State Department of Health, acting in accordance with the Public Health Law, has examined and approved the sewage disposal facilities at each of the construction areas and has general supervision over the operation of the plants. Permits issued for the discharge of sewage plant effluent have specified that a residual of 0.5 p.p.m. of chlorine be maintained therein. Suitable records are

kept by the field engineering staff of the Board of Water Supply of the number of doses of sewage applied to the filter beds, the quantities of sewage, the amounts of chlorine used, and other pertinent operating data. Samples of water are collected weekly from points on the streams above and below the construction areas, as well as samples of effluent from the sewage treatment plants and of drainage water pumped out of the tunnels. This work together with general supervision of the sanitary work occupies much of the time of four engineers of the Board of Water Supply. The samples are analyzed, as requested, for physical and chemical characteristics and bacterial content in the laboratories of the Department of Water Supply, Gas and Electricity. Weekly reports on these analyses are prepared for the information and guidance of the Chief Engineer of the Board of Water Supply and his staff. In addition, where the work is in the drainage areas of existing reservoirs, some supervision is given by local representatives of the Department.

Turbid Drainage Water. Ground water has been encountered during the excavation of the shafts and tunnels. In general, the quantities of water pumped have been relatively small, but at times, for short periods, high rates of flow have been experienced. All of this water has been pumped out of the excavations and discharged into nearby water courses or reservoirs.

The control of dust during the operations of drilling and mucking in rock excavation has required the use of water for drills and to wet down muck piles after blasting. This water together with infiltration from the surrounding masses of rock has been quite turbid as pumped from the excavations, due to suspensions of rock flour and, at times, cement grout. At five locations rock is crushed for concrete aggregate. The washing of this crushed stone results in considerable volumes of turbid water. Surface drainage from the construction areas and spoil banks is another source of turbid water. Not only is all of this water turbid to varying degrees, but it also may carry some pollution in spite of precautions taken to reduce the chances for contamination. Treatment with chlorine has been used to control bacteria. A study of the records from three typical chlorinator installations for the year 1939, shows average chlorine applications varying from 1 to nearly 10 p.p.m. required to sterilize the water pumped from the excavations.

As a rule, drainage water has been passed through settling tanks or basins to remove as much as practicable of the finely divided suspended matter. The water has then been treated with chlorine to destroy any bacteria that may have been present. In six areas, the entire flow of the adjacent streams has been treated with chlorine to guard against contamination of the water in reservoirs farther downstream.

Disinfection of Tunnel Muck. At two of the construction sites, rock excavated from the shafts and tunnels has been dumped in fills or spoil banks in shallow areas along the shores of existing reservoirs. To guard against pollution of the reservoir water this muck has been sprayed with a chlorine solution. Some of the spraying has been done as the muck was dumped into the trucks from the storage hoppers, and the rest has been done in the truck after loading. By experiment

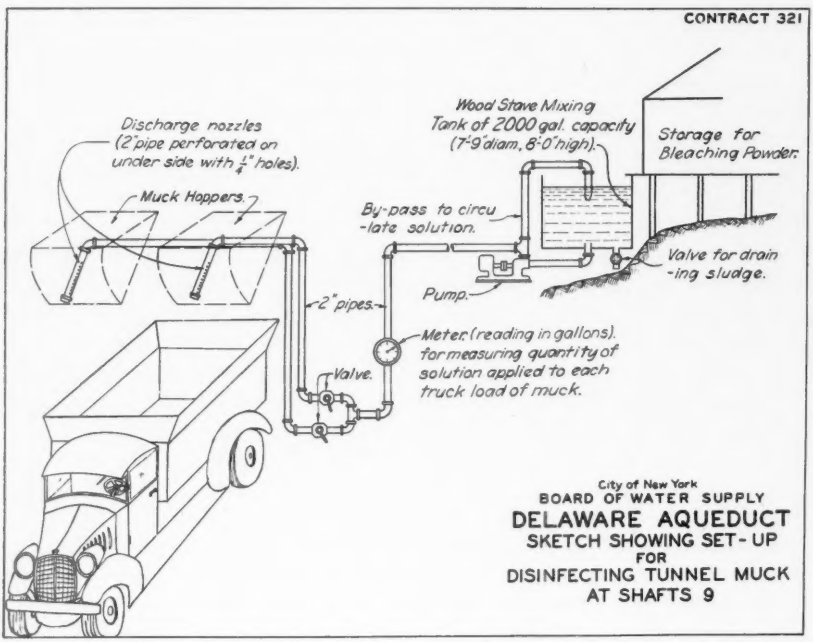
CONTRACT 321
Discharge nozzles (2" pipe perforated on under side with 1/4" holes).
Muck Hoppers.
Wood Stave Mixing Tank of 2000 gal. capacity (7'-9" diam., 8'-0" high).
Storage for Bleaching Powder.
By-pass to circu-late solution.
Valve for drain-ing sludge.
Pump.
2" pipes.
Meter (reading in gallons) for measuring quantity of solution applied to each truck load of muck.
Valve.
City of New York
BOARD OF WATER SUPPLY
DELAWARE AQUEDUCT
SKETCH SHOWING SET-UP
FOR
DISINFECTING TUNNEL MUCK
AT SHAFTS 9

FIG. 5.—DIAGRAMMATIC SKETCH OF SET-UP FOR DISINFECTING TUNNEL MUCK.

it has been found that about 4 gal. of a solution containing 1% of available chlorine will properly treat a cubic yard of fairly dry broken rock. If the rock is wet, a somewhat smaller quantity of solution may be adequate. The general sizes of the rock fragments and the resulting surface areas and voids have some bearing on the necessary quantity of chlorine solution. A diagrammatic sketch of one muck-disinfecting outfit is shown in Fig. 5, and the process of disinfecting a truckload of rock is shown in Fig. 6. The chemical storage, handling,

FIG. 6.—DISINFECTING TRUCK LOAD OF ROCK.

and mixing equipment and the tanks and piping have been housed in heated buildings to prevent freezing. In cold weather, warm water has been used to keep the solution at a temperature of about 70° F. The solution has been passed through a meter of acid-resistant construction, and the quantity used on each truckload of muck has been

determined from an indicator dial similar to those on pumps at gasoline filling stations. The chemical has been purchased in two powder forms, one as CCH, an industrial calcium hypochlorite containing 70% of available chlorine, and the other as commercial bleaching powder containing 35% of available chlorine. Both have been delivered in metal drums.

Cost of Sanitary Services. The cost of building nine of the sewage-treatment plants varied from $13,600 to $22,750 each, including two chlorinators at each plant, but exclusive of the cost of sewers, land, and the Board's engineering.

The cost of operation and maintenance of all of the sanitary services, based on the contractors' lump sum bids and the contract times for completion for 18 construction contracts, is equivalent to an average of $395 per month for each construction area, or 0.5% of the total amount of the construction contracts. This sum is exclusive of the cost of chlorine and the cost to the City of New York for engineering and laboratory charges.

Miscellaneous. The construction specifications have required the contractors to clean and disinfect all toilet rooms, lavatories and wash rooms at frequent intervals, to sweep, wash, and otherwise keep offices and buildings clean, to remove garbage in tight receptacles daily, and promptly to collect and dispose of all other putrescible matter and rubbish about the works.

From the fall of 1936, when construction work on the Delaware Aqueduct was started, to July 20, 1940, over 83 million dollars in value of contract work has been done. During this period there has been no outbreak of water-borne disease that could be attributed to the construction work. This record testifies to the good work of the contractor's men in charge of sanitary works and the faithful supervision rendered by members of the engineering staff of the Board of Water Supply. This result is ample justification for the trouble and expense involved in the program of sanitary precautions adopted.

Acknowledgments. The author is indebted to Charles M. Clark, Chief Engineer; Roger W. Armstrong, Deputy Chief Engineer; Frank H. Nowaczek, Assistant Engineer; and other members of the engineering staff of the Board of Water Supply, for assistance in preparing this paper.

DISCUSSION

RAYMOND E. FLINT.* From the contractor's viewpoint, the sewage disposal plants as constructed and equipped on the Delaware Aqueduct have operated very satisfactorily, requiring a minimum of maintenance. This has been largely due to the excellent effluent delivered to the filter beds where the maximum of maintenance can be expected in keeping the surface of the filters clean. An occasional light scraping has sufficed to keep the beds in very good condition. The only consistent repair item in these plants has been the replacement of cracked tile pipe where it has entered or left concrete structures. This has been due to unequal settlement of the two units. The experience gained along these lines suggests that a more elastic jointing material other than cement mortar in joints adjacent to such structures would be advisable.

The chlorination in conjunction with the sewage disposal plants has been easily controlled to the satisfaction of all concerned. The only perplexing problem arising in the chlorine houses has been that of heat during cold weather. The houses containing the chlorinators are simple one-room buildings, usually insulated and requiring very little heat to maintain the desired operating temperatures. However, in addition to heating the room it has been mandatory to heat the feed water for the chlorinating machine in order to prevent freezing. In most case these plants have been in isolated areas of the shaft sites where they are inaccessible to central heating plants used in other buildings. It was hoped that it would be possible to operate by heating only one of the two factors involved. Experiments very quickly proved the inadvisability of such a procedure as the temperature required of the room to prevent the freezing of the feed water (or vice versa) produced maxima detrimental to safe operation. Since these buildings require an inspection but once in twenty-four hours, it became advisable to use heating apparatus that would require a minimum of attention and guarantee a continuous output of heat. While small oil burning stoves are suitable to heat the buildings, they are entirely inadequate to heat water. They also present a definite fire hazard as has been learned from their usage. Such a potential possibility precludes the use of such apparatus when other means are available

*Engineer, Frazier-Davis Construction Co., St. Louis, Mo.

because of two reasons: (1) the loss of the plant prevents treatment of the waste waters involved and (2) a fire in such a plant would release the liquid chlorine stored therein causing unpredictable damage. Such factors have favored the use of electrical heaters. Ordinary "bar or strip" heaters have been used to heat the buildings, while immersion heaters have been used to heat the feed waters. Such units, thermostatically controlled, have proved entirely satisfactory from all points of consideration except that of operating costs. This single exception is dependent upon power rates and is an individual proposition. However, due to this factor, recent changes have been made in isolated plants to reduce this cost. Installations of small hot-water furnaces equipped with oil burners have been made. Inasmuch as these units have not seen active service of any length of time, it is impossible to ascertain if the desired result has been accomplished. Such units are to heat both the building and feed water.

The disposal of waste waters has produced unexpected difficulties due to the turbidity of such waters. The turbidity is practically entirely due to the drillings and dust from tunnel operations and fortunately, is inorganic matter. The intensity of this load has been a maximum due to the small amount of water encountered in the tunnels. As a result the main portion of the waste waters has come from the drills. This water has been collected immediately in the heading and relayed by pumps to the surface. The gradation of such rock drillings has ranged from a coarse grit to a flour. The rate of sedimentation has therefore varied and deposition has occurred in pump sumps, weir boxes and detention basins in the chlorinating plants. This deposition in such units was anticipated, but not in the amounts that have occurred. The discharge lines transporting this pumpage have not been equipped with provisions for by-passing the flow into stand-by units to allow alternate operation and cleaning. Such deposited material has had to be removed while the units have been in operation. While such a procedure maintains the plant, it is not the best to be desired, as this causes heavy loads of turbid water to be discharged from the system. Aside from clogging the detention basin in the chlorinator plants, this material has handicapped the control of chlorination due to its interference with the routine colormetric tests used in this work. As a result, field control had to be placed under the operator's judgment in many instances, and the results of the weekly

analyses as made by the Board of Water Supply used to verify or correct the rate of application of chlorine. While much of this turbidity has been retained, as is indicated in the units previous to chlorination, additional quantities have been entrapped before final discharge. However, the experience gained in these plants indicates that it would be advisable to attempt a more complete removal of this material before chlorination.

The disinfection of tunnel muck has been largely a problem of suitable application of chlorine and the provision of a temporary plant sufficiently large yet reasonable in cost of erection and operation. Cold-weather operation has made it advisable to make these plants as compact as possible. As a result, chlorine solution has been applied to a loaded truck which stops under an overhead spray, the spray being housed in with the mixing tanks and balance of the plant so as to prevent freezing. The application and control of this solution has been done with pumps and meters of special construction so as to handle the highly corrosive material. The tanks for storing and mixing the solutions are of timber. They have stood up fairly well, except the bottoms, for slightly better than a year. A concrete slab has recently been placed in the bottom of these tanks to assure their lasting until the plant can be shut down. The chemical used has been commercial bleaching powder and was decided upon after a comparative study of the costs of other available materials.

The maintenance of an adequate water supply has been largely a matter of quantities rather than treatment. From the contractor's point of view, there are three distinct water demands: (1) that of drinking water and water for sanitary purposes; (2) the demand of the contractor's plant, such as water for cooling purposes, concreting, and washing crushed stone; and (3) the fire demand. It will be recognized that all three demands can be supplied from three different sources or from only one or several combinations. Accordingly, water supply systems have been established in compliance with the sources of supply on hand. In all cases, it has been found that satisfactory treatment can be obtained by feeding a sodium hypochlorite solution into the supply pump discharge.

The other miscellaneous phases of sanitary work, such as maintenance of locker rooms and offices and the removal of putrescible wastes, have proved to be a concern of janitor service only.

PURCHASED POWER FOR WATER WORKS PUMPING STATIONS

BY WELLEN H. COLBURN*

[*Read January 16, 1941.*]

One may well ask, in presenting a paper of this type, why the question of the relative advantages of different forms of power for pumping water comes up perennially for discussion since factors frequently not related either to economy or facility often control in determining the selection of the power source. In dealing with the general power user, I have yet to find a case where the user has not enthusiastically acclaimed the advantages of purchased power and acknowledged his preference for it for his own use, although at the same moment he may be earnestly arguing some basis of economy to justify his installing or continuing the use of some other form of power. Even in pumping water or sewage, first consideration seems to turn to purchased power as the standard of service and of cost. This is because of all costs, it alone is determined in advance from known conditions.

In approaching this subject, I must perforce reflect my own observations and experience within the Boston Edison Company's service area. These are affected, of course, by its rate schedules and service policy. Comments herein may not apply, therefore, in areas served by other companies. However, since fundamental concepts, recognized by all Public Utility Commissions, underlie both rate structures and service policies of all power companies, the observations herein made will usually apply subject to the effects of local conditions. For example, rates for purchased power will, in general, be less in a metropolitan area than in rural sections; and capital expenditures by the service company to assure continuity of service can be more readily justified by the service company in the more thickly settled areas also.

Since so many factors bear upon the selection of a source of power for water pumping, it would be presumptuous for me to sug-

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gest that purchased power should always be the choice even though it may be (in fact, usually is) justified by sound economics. I shall not have the temerity to recite here the elements of real cost to the citizen-taxpayer whose ultimate interest should be your first concern. You know them all too well. Recognition of these costs is all too frequently eliminated in making a selection of the power source. I speak as a taxpayer. Suffice it to say that these neglected cost items usually relate to attendance and related expenses and to what in industry would be known as capital investment costs, interest, taxes, insurance, and amortization. I have yet to find a case where, with proper design, at my company's rates for purchased power, and with all elements of cost properly included in the analysis, economy indicated installation of another form of power.

As confirming the foregoing argument, I recollect a discussion in your meeting here in September in 1938 in which the statement was made that the "controlling factor (in the selection of power) is never comparative operating costs but other unrelated considerations." With the above comments, I pray to be permitted to forego the usual controversy over relative economy of different sources of power per kilowatthour or per million gallons pumped and to be allowed to treat of other not unrelated considerations.

Before I go any further, let me remark that I desire to throw no stones at steam power or internal combustion engine power for pumping or for any other service. You know better than I do the factors affecting operation of such power sources. Suffice it to say that with proper care and attention, they, too, will pump the required quantities of water.

City Water Pumping. Water requirements in urban areas are usually supplied from a system of reservoirs. The daily use of water is of such magnitude that continuous pumping to the mains is indicated. Consideration was formerly accorded steam engines as a source of power. These are applied directly to the pumps. High first cost, high maintenance and attendance cost, the additional investment necessary to provide boiler space and equipment, coupled with relatively poor steam and hence fuel economy as against low electric rates, are operating practically to eliminate steam power from consideration for this service today.

The reliability, availability, and continuity of electric power serv-

ice in urban areas are assured by a multiplicity of generating stations and interconnections for interchange of power between adjacent and even distant power companies. For example, my company has two interconnected fuel-burning generating-stations located on tidewater at Boston and at Weymouth, and a small hydro-electric supply. Construction of a third fuel-burning station is being currently undertaken to the north of Boston for early completion. It also has connections over four separate routes with the New England Power Company to the south, west, and north by which power can be exchanged, and many alternate routes for load transfer exist within the company's own system. Underground transmission lines and services afford, to a considerable extent, a further insurance. The relatively high degree of reliability of electric equipment is not questioned.

The smaller quantity and size of power and pumping equipment with purchased electric power indicates less land area and a smaller building required. These, with the lower first cost, thereof, may prove to be important considerations in congested urban areas, while the elimination of noise and smoke is almost imperative.

Much the same considerations apply to the use of internal combustion engines for this service. Space requirements and first cost are commensurate with steam. Noise may present a real problem, and oily products of combustion aggravate the already serious problem presented by the products of imperfect combustion discharged into the air by numerous oil burners.

City water-pumping service is usually of long-hour duration at fairly constant loads. Purchased power rates for large pumping service of this type are usually of a two-part type comprising a demand charge upon the maximum rate of power used and a charge for total energy consumed. Such a rate can yield pumping costs of from one cent to one and one-quarter cents per kilowatt-hour. This type of rate also includes the lighting and incidental power ordinarily purchased independently at higher rates where an integral power plant is used.

Town Water Pumping. In the field of water pumping in suburban and rural areas, steam power, because of its high first cost and operating costs, is not usually considered. Because of this, consideration is given to and selection lies between internal combustion engines, usually Diesel, and purchased electric service.

These pumping stations in sparsely settled areas sometimes are

situated so remotely that the cost of extending power lines to them cannot be justified by the power company upon the anticipated income. In such situations, the pumping plant may have to bear a part of the cost of a service extension. Such a consideration has resulted in some instances in selection of internal combustion engines as the power units. This selection under such conditions may appear economically sound. Changing conditions frequently reduce the service cost, and service becomes available. Unfortunately, the higher investment in engine equipment cannot be readily liquidated and the advantages of purchased power realized.

The same factor of service installation and extension cost in lightly loaded suburban and rural areas dictates overhead line construction. This being subject to the hazards of nature suggests the desirability, if absolute certainty of power availability is essential, of providing a standby unit. This may be most economically provided by a gasoline-engine-driven pump. This type of engine, if of the automotive type, is low in first cost and reasonably dependable. Since it will be used rarely and then for only short periods of time, its relatively short operating life and its high fuel cost for this service can be accepted. A similar provision is mandatory with a Diesel plant unless a duplicate unit is provided. Use of the standby unit will be much greater in this case and its short life and high fuel cost more onerous.

Suburban and rural water pumping may be from ponds or reservoirs, from shallow pits, from artesian wells, or from wells of the gravel-packed type. Pumping is usually to a system of mains regulated by one or more reservoirs or standpipes. The pumping head and volume may vary to some extent, depending upon water requirements and system water level, and the power requirements will vary in like proportion. Also, population trends and more abundant living necessitate provision for future growth.

In this type of station, purchased electric service can show great economy. The most efficient pumps and pump speeds can be selected for direct motor drive in either horizontal or vertical position, without expensive and loss-contributing gears, to provide the simplest and least costly arrangement. These pumps can be selected of a size to meet current requirements with best efficiency and with provision for other or duplicate units to meet heavy draw-down and normal growth. Such an arrangement permits limitation of early investment to needs, re-

stricts the size of bond issues, and keeps down the interest charges until such time as a larger plant is required.

With such a plant, and purchased service, the most reliable and fool-proof regulation becomes possible through the use of automatic controls. By a little consideration of pumping needs, the optimum economy in power cost can be secured. The accompanying graphic load charts illustrate the operation of such a station under disadvantageous and under optimum adjustment for cost of power. Figure 1 shows short duration peak loads with three pumps operating, with

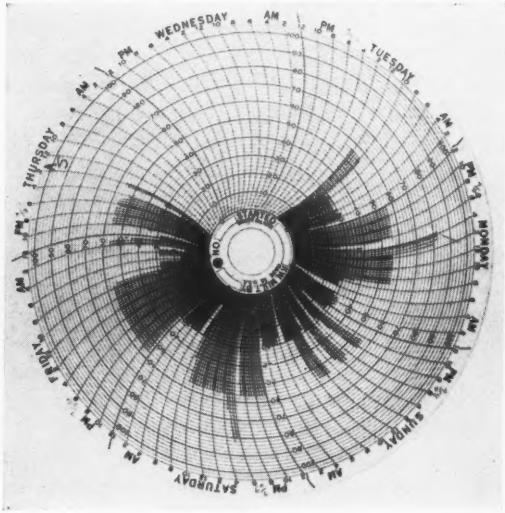


FIG. 1.—ELECTRIC POWER RECORD OF THREE-UNIT PUMPING STATION, SHOWING OCCASIONAL HIGH LOADS OF SHORT DURATION WITH MANY PERIODS OF SINGLE-UNIT OPERATION.

longer periods of two pumps or only one operating. Figure 2 shows an extended period of peak load pumping following an all-night shutdown. This kind of operation establishes an unfavorable load factor (ratio of kilowatthours to kilowatts of demand) and a higher power cost than necessary. Figures 3 and 4 show the load characteristics of the same plant adjusted to operate under the most advantageous conditions of pumping. Twelve months' operation with the load dis-

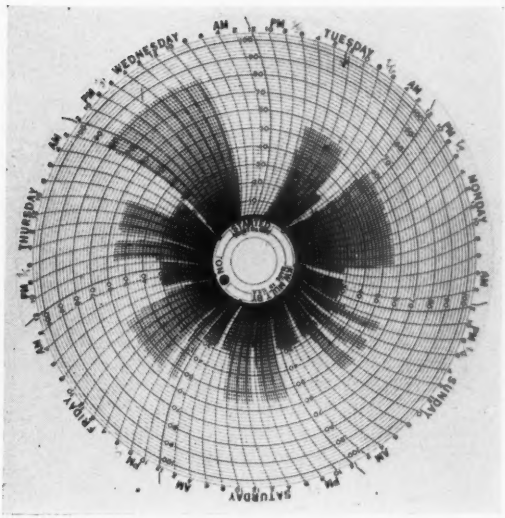
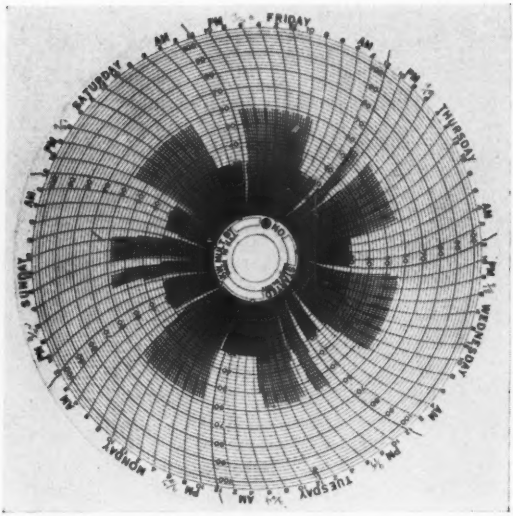
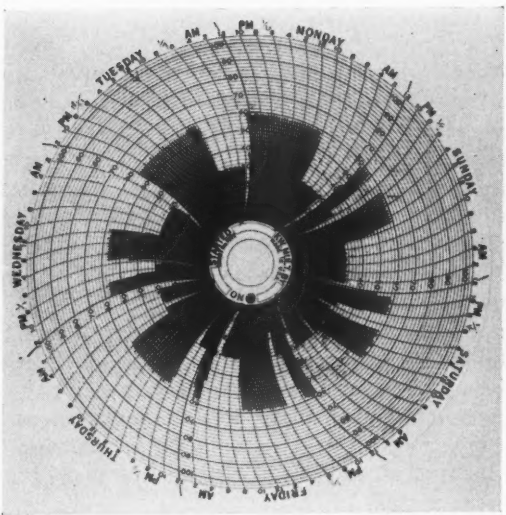


FIG. 2.—ELECTRIC POWER RECORD OF THREE-UNIT PUMPING STATION SHOWING ONE EXTENDED PERIOD OF HIGH LOAD AND SEVERAL PERIODS OF SINGLE-UNIT OPERATION.

played by these charts resulted in an average rate for purchased power of 1.47 cents per kilowatthour. In a somewhat similar installation where close control proves not so practicable, a different type of rate applies; but the cost is still only 1.55 cents per kilowatthour. Twelve small pumping plants in my company's area on yet a third type of rate show an average cost of from 1.42 to 2.86 cents per kilowatthour with an average of 1.58 cents. The fact that the average of all is near the low figure demonstrates how the cost drops with increasing requirements for water pumped.

I feel sure that there is no need for me to attempt to prove that electric-motor drive with purchased power demands less attendance and supervision and shows a lower repair cost than any other form of power. These items, particularly attendance, with other forms of power may involve other cost items, such as a roving operator, car mileage, or the cost of transient observation by members of the police or fire department staffs.





FIGS. 3 AND 4.—ELECTRIC POWER RECORD OF THREE-UNIT PUMPING STATION SHOWING OPERATION ADJUSTED TO AVOID OPERATION OF THIRD UNIT EXCEPT FOR EMERGENCY.

As an illustration of some of these points, some statistics secured from town records and reports of water department superintendents, with comparative purchased power cost estimates, are appended to this paper. These were gathered and prepared by J. W. Wattles, 3rd, for a paper presented before the Power Engineers Association. It just so happens that in each case these stations were powered by Diesel engine units.

In spite of my statement at the beginning of this paper, I have been unable to keep entirely away from it and have fallen into the economics of the question. Since I'm in it, let us note the cost items applicable to any plant whether making or purchasing power. These are:

1. Fuel and related costs such as ash disposal.
2. Water for boiler feed or for cooling.
3. Lubricants.
4. Repairs.
5. Attendance (all labor and related services) and supervision, including accounting and purchasing services.
6. Miscellaneous and supplies of all descriptions.
7. Interest.
8. Insurance—breakdown, and use and occupancy, whether purchased or not.
9. Taxes—lost, if any.
10. A reserve to replace the equipment when that becomes necessary.
11. Purchased power.

Selection of purchased power for the pump drive keeps all of these items to the smallest dimensions with the exception of item 11. Because of the functioning of rate structures, even this item will incorporate a smaller cost for the incidental power and lighting than if there were an integral power plant. The combination of these cost items that results in the lowest total with the best service is the best for you and the one desired by the citizen-taxpayer.

In conclusion, purchased power offers the following advantages:

1. Low investment.
2. Optimum plant and equipment design for efficiency of operation and use of capital.
3. Ready provision for growth without major problems.
4. Little maintenance.
5. Minimum attendance and supervision.
6. Commitment to a relatively small loss should it become desirable to abandon a station and locate elsewhere.

7. A high degree of availability and reliable service.
8. The establishment of an increased real base for taxation in the service company's investment.

APPENDIX

STATISTICS OF COMPARATIVE OPERATING COSTS.
PREPARED BY J. W. WATTLES, 3RD.

Station A

Investment	
(1932) Diesel and gasoline units	$10,525
(1938) Circulating system	531
Operating Costs	
1938—114,694,000 gal. Dynamic head—230 ft.	
Fuel—Diesel 9,853 gal. at 6c	$ 591
Heating 1,465 gal. at 6c	88
11,318 gal.	
Labor—Engineers	1,820
Oil, Waste, etc.	543
Gasoline	87
Insurance	103
Electric light	35
	$ 3,267
Overhead on $11,000 investment	
Interest 4% + Depreciation 5% = 9%	990
Total cost with Diesels for 1938	$ 4,257
Operating Costs if Electric	
Power*—	
$\frac{114,694,000 \times 8.34 \times 230 \times 0.746}{33,000 \times 60 \times 0.75 \times 0.87} = 127,000$ kw-hr.	
Boston Edison Company—Rate J	
30-kw. demand—127,000 kw-hr.	$ 1,994
Labor	300
Repairs, Supplies, Heat, and Light	150
	$ 2,444

*Gailons x 8.34 = lb.
lb. x ft. = ft. lb.
ft. lb. ÷ 33,000 x 60 = hp.-hr.
hp.-hr. x 0.746 = kw.-hr.
0.75 = pump efficiency.
0.87 = mechanical efficiency.

Investment—1 electric and 1 gasoline unit $8,000 at 9%	720
Total Annual Cost with Electric Power	$ 3,164

Station B

Operating Costs—Diesel Plant	
1938—83,474,000 gal. Average head 220 + 20 ft. friction	
Fuel, 7,500 gal. No. 2 oil at 6c	$ 450
Lubrication, 90 gal. at 60c	54
Electric Power, 8,640 kw-hr.	268
Repairs and supplies	300
Attendance—part time + extra mileage	1,000
	$2,072
Overhead on $10,000 investment	
Interest 4% + Depreciation 5% = 9%	900
	$2,972
Operating Costs if Electric	
Power—	
$\frac{83,474,000 \times 8.34 \times 230 \text{ ft.} \times 0.746}{33,000 \times 60 \times 0.75 \times 0.85} = 95,000$ kw-hr.	
Boston Edison Company—Rate J	
30-kw. demand—95,000 kw-hr.	$1,703
Heating building	50
Attendance	200
Repairs and supplies	100
	$2,053
Investment—New gravel-packed well with deep well turbine pump $5,000	
Interest 4% + Depreciation 5% = 9%	450
Total Annual Cost with Electric Power	$2,503

Station C

*Operating Costs**—Diesel Plant		
1938—214,723,000 gal. pumped. Average dynamic head—305 ft.		
Fuel oil 25,777 gal. at 6c		$1,546
Lubrication 208 gal. at 40½c		84
Repairs and Attendance		
Labor—1 engineer	$1,600	
Auto travel 50 miles per day at 5c	910	

*1938 Operating Costs obtained from Superintendent of Water Department.

Repairs and maintenance	100	2,610
		$4,240

Investment — *Building* $6,000; *Equipment* $10,000; *Total* $16,000

Interest 4%

Depreciation 5%

9% of $16,000 — 1,440

Total Cost with Diesel for 1938 — $5,680**

Operating Costs if Electric

Power—

$$\frac{214{,}723{,}000 \times 8.34 \times 305 \times 0.746}{33{,}000 \times 60 \times 0.70 \times 0.87} = 338{,}000 \text{ kw-hr.}$$

Boston Edison Company—Rate G1—45-kw. demand	$4,090
Labor and auto travel	1,200
Repairs and supplies	50
	$5,340
Investment — *Building* $2,600; *Equipment* $4,400; *Total* $7,000 at 9%	630
	$5,970

Station D

Operating Costs—Diesel Plant

1938—104,738,000 gal.* Average head—252 ft. + 45 ft. friction

Fuel, light, power	$1,011*
Repairs, engines, wells, pumps (average)	262*
Attendance, regular	1,500*
Extra man overtime	300
Upkeep and repairs	37*
Equipment and supplies	72*
	$3,182
Investment in Pumping Station 1926	
Estimated at $25,000 at 9%	2,250
Total Costs with Diesels for 1938	$5,432

Operating Costs if Electric

Average head 250 ft. + 20 ft. friction

Using vertical deep-well turbine pump in the new well

Power—

$$\frac{104{,}738{,}000 \times 8.34 \times 270 \times 0.746}{33{,}000 \times 60 \times 0.73 \times 0.86} = 141{,}500 \text{ kw-hr.}$$

**Note: This does not reflect the increased cost of electric service to other stations of this water system.
*From Town Report.

Boston Edison Company—Rate J		
25-kw. demand—141,500 kw-hr.		$2,065
Attendance		300
Maintenance and supplies		100
		$2,465
Investment if Originally Electric		
With 1 motor and 1 gasoline engine	$18,000	
1 deep well turbine pump	3,000	
	$21,000 at 9%	1,890
Total Cost if Electric		$4,355

Station E

Investment—Diesel Plant		
Building	$18,493	
Equipment	18,741	
	$37,234	
Operating Costs—Diesel Plant		
1938—82,096,000 gal.* Average head 210 ft. + 15 ft. friction		
Fuel, 7,059 gal. at 5.24c		$ 370*
Attendance, 1 engineer		1,500
Lubrication, 76 gal. at 60c		46*
Heating 6,688 gal.		366*
Maintenance, supplies, and telephone		1,073
		$3,355
Interest 4% + Depreciation 5% = 9% of $37,234		3,350
Total Cost with Diesels for 1938		$6,705
Operating Costs if Electric		
Average head—208 ft. + 7 ft. friction		
Power—		
$\frac{82,096,000 \times 8.34 \times 215 \times 0.746}{33,000 \times 60 \times 0.73 \times 0.86} = 88,200$ kw-hr.		
Boston Edison Company Rate J		
15-kw. demand—88,200 kw-hr.		$1,272
Heating portion of building		100
Attendance		300

*From Town Report.

Maintenance, supplies, and telephone				200
				$1,872
Investment if Electric	*Building*	*Equipment*	*Total*	
	$9,000	$11,000	$20,000 at 9%	
				1,800
Total Annual Cost if Electric				$3,672

DIESEL DRIVE FOR WATER WORKS PUMPING STATIONS

BY JOEL W. REYNOLDS*

[*Read January 16, 1941.*]

The Diesel engine is the most efficient prime mover that has ever been devised. The load on a waterworks pump is almost always steady and non-fluctuating. The combination of the typical waterworks pumping-load with the inherent high efficiency of the Diesel engine results in a low cost of pumping that has yet to be approached by any other source of power. The application is usually ideal; however, like any other piece of machinery or equipment, the conditions of service pertaining to the individual pumping job should be carefully studied by competent parties to ascertain whether or not Diesel power is a feasible and profitable investment to consider as the solution to the problem at hand. I do not say that Diesel power is correct for every pumping job, for in many cases it is not, but I do say that no careful management can afford to ignore its possibilities. Diesel engines have been widely publicized these last few years with their widespread usage in streamlined trains, trucks, busses, etc., and they have caught the fancy of the general public. One often hears today the remark made, "that Diesels are the coming thing". Most people fail to realize that the Diesel engine arrived many years ago, and that it has been built in this country for nearly 40 years.

Development. As you probably know, the principle was conceived by Dr. Rudolph Diesel in Germany in 1892. It was brought to the United States in 1896 by Adolphus Busch, of Budweiser Beer fame, who saw its tremendous possibilities. The first unit was built only about 50 miles from Boston at the plant of the old Providence Corliss steam engine of Hughes & Phillips Company. The first unit could hardly be called a success, as the builders were used to building engines for steam temperatures, pressures and tolerances, and had absolutely nothing to guide them so far as any existing prior experience was concerned. The first unit did not seem to run for any great length of time. More experimentation followed and resulted in the

*Gustavo-Preston Company, Superior Diesel Engines, Boston, Mass.

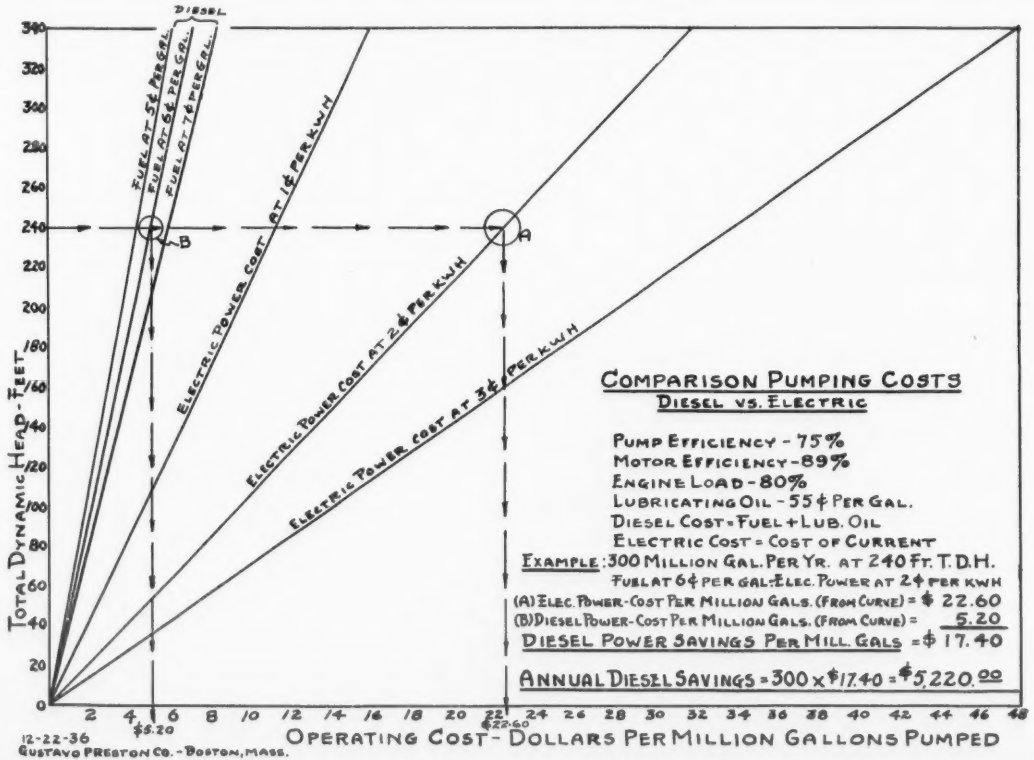
first commercial Diesel engine being exhibited at the World's Fair in St. Louis in 1904 by Mr. Busch. Incidentally, that first unit is still in service occasionally today.

There has been no appreciable increase in the thermal efficiency of modern Diesel engines over those built in the early 1900's. Improvements have been practically all of a mechanical nature with the use of better machine methods, greatly improved metals and alloys, simplification, and the natural evolution of improvement in design from field experience and research. Economy in fuel consumption of the modern engine is only about 3 to 5% better than that obtained in the early engines, and as no prime mover has been developed to approach its thermal efficiency in nearly 40 years, the obsolescence factor can be practically disregarded. This is evidenced by the many Diesel units in daily service that are 30 years or so young, and still as efficient in overall cost of operation as many modern units. All parts of a well designed Diesel that are subject to wear are replaceable. As long as a Diesel is kept in repair, therefore, it will last and give efficient service indefinitely. Its salvage or resale value is higher than for any other type of prime mover.

A point in which the Diesel engine has a decided advantage over any other prime mover is that the small unit of 3-hp. capacity (and they are built that small) is very nearly as efficient as the large unit of 10,000-hp.; therefore, in a Diesel the small hp. user can obtain the same high efficiency as the largest hp. user.

Operating Costs. With the widespread installation of household and industrial oil burners, and Diesels, the availability of a constant and readily obtainable source of fuel supply is unquestioned. As to cost, fuel-oil prices fluctuate along with, and follow other fuel prices closely. Therefore, costs of other sources of power affected by any kind of fuel cost would, naturally, increase along with any marked increase in Diesel fuel-oil cost.

Fuel-oil consumption is a major factor entering into Diesel pumping costs; however, fluctuations of several cents per gallon do not materially add to the cost of pumping a million gallons of water in comparison with other sources of power. Assuming a pumping head of 240 ft., which is common in many waterworks, and with comparable pump efficiency, a jump from 5¢ per gal. to 7¢ per gal., or a 40% advance, increases the pumping cost per million gallons for fuel


DIESEL
FUEL AT 5¢ PER GAL.
FUEL AT 6¢ PER GAL.
FUEL AT 7¢ PER GAL.
ELECTRIC POWER COST AT 1¢ PER KW-H
ELECTRIC POWER COST AT 2¢ PER KW-H
ELECTRIC POWER COST AT 3¢ PER KW-H
A
B
TOTAL DYNAMIC HEAD - FEET
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40 42 44 46 48
$5.20
$22.60
OPERATING COST - DOLLARS PER MILLION GALLONS PUMPED
COMPARISON PUMPING COSTS
DIESEL VS. ELECTRIC
PUMP EFFICIENCY - 75%
MOTOR EFFICIENCY - 89%
ENGINE LOAD - 80%
LUBRICATING OIL - 55¢ PER GAL.
DIESEL COST = FUEL + LUB. OIL
ELECTRIC COST = COST OF CURRENT
EXAMPLE: 300 MILLION GAL. PER YR. AT 240 FT. T.D.H.
FUEL AT 6¢ PER GAL: ELEC. POWER AT 2¢ PER KWH
(A) ELEC. POWER - COST PER MILLION GALS. (FROM CURVE) = $ 22.60
(B) DIESEL POWER - COST PER MILLION GALS. (FROM CURVE) = 5.20
DIESEL POWER SAVINGS PER MILL. GALS = $ 17.40
ANNUAL DIESEL SAVINGS = 300 x $17.40 = $5,220.00
12-22-36
GUSTAVO PRESTON CO. - BOSTON, MASS.

from $4.00 to $5.60, or by $1.60. An increase in purchased power cost of 40% from 2¢ to 2.8¢ per kw-hr. increases the pumping cost per million gallons from $22.60 to $27.10, an advance of $4.50.

With present-day lubricants and pressure lubricating-oil systems, this item entering into the costs of Diesel pumping is very low. Taking the same example of a 240-ft. head, it amounts to 31¢ per million gallons when the cost of lubricating oil is 55¢ per gallon. Many good lubricating oils are available and suitable for Diesel use at a cost lower than that just stated.

Maintenance and Repair Costs. Maintenance and repair costs raise a question on which considerable misinformation is prevalent. This is often the cause of much controversy in discussing proposed Diesel costs. A Diesel engine is a piece of mechanical equipment, similar in principle to the familiar gasoline automobile engine but of greater simplicity. It should be maintained in good condition if high maintenance costs are to be avoided, just as the automobile engine. Operators that neglect their automobile engines have unnecessarily high maintenance and repair costs, and so do operators that neglect their Diesel engines. Like an automobile engine, there are always signs of need of attention and servicing, which in practically all cases can be foreseen and attended to when convenient. For example, if you find that your automobile consistently knocks on the hills, you realize you need a carbon cleaning and perhaps a valve job, but you do not rush to the nearest service station at the first knock. You may run even for a month or two before you have the job done, but if you neglect it too long, it will cost more to have the trouble rectified and the engine put in first class shape.

In a Diesel engine, being an internal combustion engine, conditions are directly comparable. In 99 cases out of 100, unforeseen and sudden breakdowns and failures are definitely avoidable, and I might add that sudden failures of a Diesel, even with considerable disregard of good operating practice, are very rare. With present-day improved materials, improved machining technique, and simplified and improved design, and with care taken in the proper selection and rating of a Diesel to do a given job, maintenance costs are at a minimum and reliability at a maximum. You have but to talk with Diesel waterworks operators to have this statement supported.

Of course, maintenance and repair costs are directly affected by

the service factor or number of hours of use in a year. One station may have a 75-hp. Diesel unit which is required to operate for only 5 hours per day, while another station may have the same size of unit and pump during 15 hours per day to take care of the load. Obviously, the longer hours of use mean greater maintenance costs annually, but as longer hours of use mean more water pumped for the same size of unit, assuming the same operating conditions, the cost per million gallons should be about the same. Many stations make it a general practice to overhaul their units, say once a year or every 15 or 18 months, at which time many parts are renewed, including piston rings, which may not actually be required to be replaced so far as condition is concerned but which are replaced at that time as a matter of policy.

But, you will ask, what figure should we take or expect for maintenance? A figure of $1.00 per rated horsepower per year is often taken as a repair and maintenance cost over a long period of time, and this figure is not far from wrong. If this amount were charged and set aside each year as a maintenance fund, whether used or not in that particular year, it should be sufficient to maintain an engine in good repair for an indefinite period. Naturally, when an engine is new, there should be few or no repairs for several years. When the time comes for a major overhaul, such a maintenance fund should amply cover the cost of reconditioning. At any rate, the relative cost per million gallons pumped is very low. The impartial annual A.S.M.E. Survey of several hundred plants shows average costs of maintenance and repair well under $1.00 per rated horsepower per year. With a 75-hp. engine, at $1.00 per rated horsepower per year, or $75.00 set aside and charged as a repair and maintenance fund, and with 200 m.g. of water pumped, the cost per million gallons for this item becomes 37½¢, or about the same cost as for lubricating oil for a 240-ft. head.

The petroleum industry was among the very first to use the Diesel engine for pumping and is today one of the largest users of Diesels. Service conditions are comparable to those in waterworks in that there is a steady pumping load but the service factor is much more severe because pumping continues 24 hours per day, at or near full load, 7 days per week. There is probably no prime mover that is called upon for such severe duty as the Diesel engine in petroleum pipe-line pumping, yet statistics show that repair and maintenance

costs for these units fall well below the basic figure of $1.00 per rated horsepower per year. Where water is pumped four or five miles, petroleum is pumped 50 miles. Where water pressures are of the order of 100 lb. per sq. in., petroleum pressures are 800 lb. per sq. in. Hundreds of thousands of horsepower are delivered by Diesels in pipe-line service, and many of them have been in service continuously for 25 to 30 years, at close to full load. In the 20-year period from 1920 to 1939 inclusive, there have been installed 787,155 Diesel horsepower for petroleum pipe-line pumping, and 198,100 Diesel horsepower in waterworks pumping plants; also 202,000 Diesel horsepower for irrigation and drainage pumping plants.

Attendance. Attendance is another subject on which there is considerable misconception. Many people look upon the Diesel as a mysterious machine requiring a highly skilled, intensively trained and experienced specialist hovering over it with a wrench in one hand, an oil can in the other, and a stethoscope attached to his ears during all the time that the engine is operating. I do not claim nor recommend that any person, regardless of mechanical ability or mentality, be placed in charge of a Diesel engine or any other piece of machinery that costs much and upon which so much depends; but I do claim that a person with ordinary intelligence, a desire to learn, and with some mechanical sense can become a good Diesel operator in a remarkably short time under the tutelage of the installation engineer of the manufacturer. I do not know of a single waterworks plant in New England that upon installing a Diesel engine went outside and hired a Diesel specialist to run the plant. In almost all cases, some person or persons in the organization were instructed to operate the engines, and this is true in almost all plants in my long experience in installing units for all types of service.

In regard to having an operator in constant attendance while the Diesel is in operation, in many waterworks plants using Diesels in this vicinity the opposite is true. In many cases, some member of the department starts the unit up at the required time, sees that everything is in operating condition, locks up the station and proceeds with the outside work of the department. Go to West Acton, Harwich, Cotuit, and many other pumping stations during the day, and you probably will see the engines running if you look through the window, but try and get in, for Messrs. Clapp, Swett and Perry, respectively,

will be out working on a frozen up service pipe somewhere or laying a new main.

Many stations are equipped so that when the storage tank attains a predetermined level, a simple pressure device acts on an automatic engine stop back at the station and the unit is automatically shut down. In such cases, where the operator is not in constant attendance, it is desirable to have an automatic protective stop on the engine in case the lubricating-oil pressure falls below normal or the cooling-water temperature becomes too high for some reason. Most of the recently installed stations have these devices, as well as thermal elements hooked in with the automatic stops and attached to pump and speed-increaser gear-bearings. However, there are also many plants that operate without constant attendance and without these safety devices and that have been doing so for years with no ill effects.

At some plants the operator daily pumps up sufficient fuel oil into the day tank to run the unit long enough to fill the storage basin, before leaving for outside work, as he can tell by the static pressure present when starting and the seasonal usage, how long the unit has to run to reestablish the storage level. Not long ago, I returned to a Diesel pumping station with the superintendent late in the afternoon. We had just stepped inside the door, when the engine gradually slowed down and stopped. He didn't seem disturbed, but I asked him what was the trouble with the engine. "Oh, nothing," he replied, "I had the day tank pumped up to half past four and it has just run out of fuel." A number of successful, fully automatic pumping plants have been installed in the smaller sizes with stop and start control by a pressure or float switch.

Fixed Charges. Much time could be spent in a discussion of the pros and cons of proper fixed charges to be made against a Diesel engine plant, and, of course, such charges cannot and should not be ignored. Here again, the conditions surrounding the individual installation determine the proper overall costs including the fixed charges.

First, we should consider the proper equipment to be installed from an engineering standpoint; then we should see if the investment entailed is justified from an economic standpoint with all factors affecting the job taken into consideration. For example, in a nearby industrial plant, the only motors in the plant were two 10-hp. units attached to pumps that operated 24 hours per day, 7 days per week—

an ideal load. Power was being purchased at the low rate of 2¢ per kw-hr. with monthly bills averaging just about $230. A Diesel-driven pump was installed, having a combined capacity of the two motor-driven units. The monthly power cost with the Diesel unit dropped to just under $30, a saving of $200 monthly. The complete installation cost to the owner was $1800, who thus regained in 9 months the entire outlay and from then on made a tremendous profit from his investment. That unit was installed in 1934 and is still in service. Certainly this owner cares little about interest, depreciation, taxes and insurance. This is an isolated case, you will say, and it is, to be sure, but it is cited as an example to illustrate the necessity of figuring fixed charges, so called, depending on individual operating conditions.

When studying the value of prime movers of widely different characteristics, and where one of the prime movers of comparatively high economy has a higher first cost, it is extremely important to determine the amount of the *actual* fixed charges—as it may otherwise be estimated too high and, thereby, destroy the advantage of adopting more economical equipment that will give lowest overall operating costs over a long period of time. Investment in high first-cost hydro-electric generating-plants is often justified only by taking into consideration the low depreciation due to a long useful life.

Rough estimates of the cost of Diesel power, using the so-called straight line method of figuring fixed charges covering interest, depreciation, insurance, etc., at a rate usually assumed at about 13% annually are misleading and incorrect. Plants in operation during the past 25 years and over have definitely established the Diesel as having a long and indefinite life. Depreciation is not a cash outlay that is lost, but an estimate made in advance, in order to spread an actual cash outlay over the assumed life of the equipment. This may reasonably and conservatively be considered as 20 to 25 years for a Diesel unit. Interest on a cash outlay for the Diesel becomes less each year, as the amount invested is reduced by the depreciation charged off. Certainly, as you retire waterworks bonds from the depreciation fund, your interest charges become less by the amount of interest on the bonds retired, and full interest as at the time the plant was installed is not a constant fixed expense.

It is good operating practice to be on the safe side in charging off ample depreciation, but it is vitally necessary, when one investigates actual power costs, to avoid the use of erroneous charges on such

items as interest and depreciation. One good way for determining actual fixed charges is to add to the annual interest rate (instead of an arbitrary fixed depreciation rate) that percentage which, when placed at interest and compounded annually will accumulate the original cash outlay at the end of the assumed useful life of the equipment. On the matter of interest charges, our munificent government setting the example, interest charges on municipal bonds are now and have been at an all time low. A nearby municipality recently floated a loan of $75,000 for power plant expansion at a rate of ¾ of 1% with several banking institutions eager to extend the credit. But let us take a reasonable interest rate of 4% annually and assume a 20-year useful life of the Diesel. Let us also assume a cost of installation for the Diesel unit of $5,000 more than the first cost of installation of some other source of power. With credit allowed for retirement of bonds from the depreciation fund and consequent reduction of annual interest charges, the annual average rate of interest and depreciation is not 4% interest plus 5% depreciation, a total of 9% in straight-line calculation, but only 7.36% annually on the higher first cost of $5,000 for the Diesel; this totals not $450 per year but only $368 per year. If we pump 200 m.g. of water, this adds a cost of $1.84 m.g., while we might be picking up $15 or $25 per m.g. more in savings over some other method of pumping.

Another method, often used in determining whether a unit that is more efficient but higher in initial cost is justified, is to take all actual out-of-pocket expense—such as fuel, lubricating oil, maintenance, insurance, and interest, and any labor charges—and to subtract this value from the total cost of operation of the more costly type of power and to consider the balance available annually as the depreciation fund or that which could be used to retire the added expense of the Diesel—to ascertain if the higher first cost can be retired in a reasonable length of time. You probably will say that Mr. Waddell, at the State House won't let you keep books in that way. In fact, you wouldn't keep them like that, but it does help determine whether you should put in Diesels or some other source of power.

Insurance. In the matter of insurance, like life insurance or any other kind, the amount depends on the extent and kind of coverage that you deem advisable, but adequate protection can be secured for about ½ of 1% of first cost. Incidentally, insurance rates on Diesels have steadily been reduced, as the insurance companies have been

accumulating more actuarial experience on Diesel risks. For example, besides other marked reductions in recent years, they have recently made available one reduction as high as 50% of the base rate if the user installs a closed-jacket water-cooling system, which pays for the additional equipment in a very short time. Some users will tell you that the value of the annual inspection of their equipment by the experienced field men of the insurance company offsets considerably the cost of the premium.

Example. To cite an example of the factors to be considered in the selection of the type of plant without going into too much detail, a number of years ago, a nearby town was operating an ancient crank and flywheel condensing engine with handfired boilers of the same vintage. Fuel cost was approximately $14 per m.g., and the pump had a capacity, of 800 g.p.m., which was adequate for peak demands during fire and other emergencies and during periods when the reservoir was being cleaned. The engine had been rebored to the maximum and needed a new crankshaft; the boiler had been condemned.

First they investigated a new steam-driven plant and found that with all new equipment, which would cost about $15,000, they could not pump water at a lower cost than they were then doing with their ancient equipment at a total cost of $5000 per year. Next, they considered purchased power. In order to keep their demand cost down and to obtain a reasonable cost of pumping, it was recommended that two pumps each of 285-g.p.m. capacity be installed, operating longer hours, and that the station be made fully automatic, eliminating the attendant, and that a gasoline-engine standby-unit be installed for emergency use. A larger unit of 800-g.p.m. capacity would have entailed a demand charge on the larger motor that would have made the cost prohibitive. Under this arrangement, it was calculated that the total cost of the equipment and standby would be about $7000 and that the energy cost per million gallons would be about $26 with total annual costs of $3800. The manager and commissioners felt that this arrangement had a number of disadvantages: the small total capacity would not take care of them in emergencies; they felt they wanted a man on duty at all times while pumping; and the high cost per million gallons pumped did not fully offset the elimination of the attendant.

Next they investigated a Diesel unit of the same capacity as their present steam unit (800 g.p.m.) with a gasoline-engine standby-unit of the same size, both of which could be speeded up in emergency

to 1000 g.p.m. each. The total cost of the Diesel and gas-engine standby-installation would be $11,000. Taking into account all operating and fixed charges on the three types of plant considered on the same basis, and disregarding the obvious advantage of having adequate capacity and a man in attendance in the cases of the steam and Diesel plants, it was found that with the Diesel plant total costs would be $3000 annually and would give them a saving of $800 annually against purchased power and $2000 against steam pumping. As the Diesel plant had an initial cost $4000 more than that of the electric plant, this saving would liquidate the difference in about 5 years, besides giving adequate capacity and a permanent attendant. The Diesel was installed and has long since been returning dividends in lower-cost pumping to the town.

Mechanical Considerations. Space will not permit going into the various types of engine: two or four cycle, high speed or low speed, types of injection, etc. They all have their place, depending on conditions. There are, however, several other advantages in Diesel operation of pumping units: high efficiency at variable speeds; speed can be varied to take care of seasonal demands such as occur in many of our summer resorts; and automatic or manual speed variation can be used to follow the load on direct-pressure systems.

Diesel plants are self-contained units, unaffected by outside disturbances, such as sleet, lightning, windstorms or hurricanes. There are no standby nor steaming costs when units are not pumping—steam plants that pump only during part of the day keep boilers fired for 24 hours per day. Large fuel storage requires small space—some plants have storage adequate for a year's run and buy their fuel in lots equivalent to tank car lots when prices are lowest.

The present defense program is stressing self-contained power sources because of the obvious advantages, particularly in such vital places as water-pumping stations. Some of the New England towns and cities that have Diesel-equipped pumping stations are: Burlington, Vt.; Milford, Nashua, Plymouth, Franklin, Hampton and Salem, N. H.; New Britain, Conn.; Wilmington, West and South Acton, Maynard, Wayland, Harwich, Cotuit, Bourne, Fairhaven, Foxboro, Buzzards Bay, Plymouth, Marshfield, Duxbury, Milford, Pepperell, Townsend, Newburyport, Wareham, Onset, Needham, Waltham, Boston Metropolitan District at Spot Pond and at Braintree-Weymouth, and Springfield, Mass. (sewage stations).

DEVELOPMENT OF STOUGHTON'S WATER COLLECTING SYSTEM

BY LABAN MORRILL*

[*Read January 16, 1941.*]

Stoughton is an enterprising town situated 18 miles south of Boston on the road to Taunton and Providence or to Brockton. Before 1929, we had about 26 factories of all sorts and sizes, belonging mostly to rubber, shoe, and textile industries. We now have about eighteen of the same make and a population of 8,600. It is a nice lively town and a good place to live.

In 1892, the town of Stoughton bought out the Stoughton Water Co. for $59,130.48. The plant consisted of wells, water mains, and a standpipe. The water supply was poor and the system incomplete. The Town Committee, with the help of a consulting engineer, selected Muddy Pond as a new source of water supply. It was estimated that the new source could be developed to serve 10,000 people. Stoughton at that time only had 455 water connections against about 2,000 metered services today

Muddy Pond Supply. Muddy Pond is situated 2 miles more or less westerly from the center of Stoughton. The pond itself lies, not unlike a frying pan, in a depression surrounded by steep hills; the sides of the pan being the hills and the bottom of the pan which is 18 acres in extent containing the pond. Of the 18 acres, less than 3 acres are actually flooded; the rest is just peat bog. The handle of the frying pan is the outlet channel.

Water poured out of the base of these hills and ran in small brooks through the swamp into the pond. In developing the water supply, it was decided to build a puddle dam across the outlet channel to store the water for consumption. A well was built into the dam, and a 12-in. gravity outlet was run from this point 4300 ft. to the pumping station. When this construction was finished, it was discovered that the surrounding foothills below the pond were such a good leaching medium that it was impossible to raise the pond level and increase its storage. Hence the puddle dam was abandoned.

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Next a filter basin was built above the puddle dam, and water from Muddy Pond and two adjacent streams was passed into this new structure. The water was sent into the puddle dam well and through the 12-in. gravity main to the pumping station. This system worked well for a while, but the State Department of Public Health found that samples of the water began to show pollution. It was discovered that the two brooks, Drake's Stream and Leonard's Stream, which joined the Muddy Pond channel outlet were picking up pollution from the farm land through which they passed on the way to the Muddy Pond basin.

Use of the filter basin was, therefore, discontinued, and a 15-in. tile pipe was laid from the puddle dam well upstream 600 ft. to a point on the main channel outlet of Muddy Pond. This was necessary in order to by-pass the two condemned streams around Muddy Pond. At the head of the 15-in. tile pipe, in the Muddy Pond channel, a collecting well was built into the channel with a series of screens graduated in size for the water to pour through and go on its way to the pumping station. After that, everyone seemed to be happy. There was enough water, and no thought was given to expanding the supply till the middle 1920's when Stoughton had barely enough water to get through a drought period.

A new and more direct channel was, therefore, trenched from Muddy Pond to the screens. This new channel was supposed to be deeper and a direct line to the 15-in. intake pipe that led to the puddle dam well. When the channel was being excavated a ridge of Fuller's Earth was found to extend across the downstream side of Muddy Pond. Digging through this ridge was a costly piece of work, and it was decided to build a wide channel rather than a deep one and to construct a basin near the screens in order to supply the storage necessary to make up for the planned depth of the channel. Stoughton now had enough water to last for some time. The town's industries were booming and the consumption of water jumped from 477,000 g.p.d. in 1928 to 734,000 g.p.d. in 1935, an increase of over 50%.

Pumping Station. During this period, the pumping station was remodeled. At that time, the station contained two duplex compound Knowles steam pumps and two boilers. These two pumps each had a capacity of 1 m.g.d. One of the boilers was removed and an addition was built onto the station. In the new building a Dean-Hill

centrifugal electric pump was set. This also had a rated capacity of 1 m.g.d. A Venturi meter was installed because the town water system had no accurate check on the quantity of water pumped to the consumers in the previous years of operations.

Deterioration of Water Quality. Also within this period complications set in (as a doctor would say). We had a fire that started in a neighboring town and, due to the absence of water mains in the streets on the outskirts of Stoughton at that time, the fire raced through the woods and across the Muddy Pond watershed before it could be extinguished. The fire burned over about 200 acres of the immediate watershed of Muddy Pond. This watershed of 500 acres had been protected by a stand of white pine and oak that had not been cut for years. After the fire, the area was exposed to the sun and the surface became a hard baked mass. Every time it rained, the water would run off and not soak into the ground. After a heavy downpour, a large amount of muddy surface water would rush down the steep hills of Muddy Pond and flow into the basin. This would cause discoloration of the water for a number of days, because the pond was small and less than 4 ft. deep. The storage was only 3½ m.g., and it did not take a large amount of muddy water to discolor the whole pond. The sudden increase in volume of the little brooks fed by springs out of the hillsides caused them to overflow their banks and cover the swamp. The swamp then got a thorough rinsing and drained into the pond carrying a nice swampy taste and smell into the water supply of Stoughton. All these troubles caused many complaints about the quality of Stoughton's water supply which was plentiful enough but not good enough. During this period, the regular samples sent to the State Department of Public Health showed that there was nothing serious or dangerous about the many and varied materials that went into the compounding of Stoughton's water. A chlorinator was mutually agreed upon to be installed in the pumping station to safeguard against possible contamination of the water. A close watch was kept on the dosage of the chlorinator by use of orthotolidine, and after a heavy rain a larger amount of chlorine was introduced into the water pumped to the town.

Watershed Improvements. Mr. Winslow, the Town Manager, decided that something should be done about Muddy Pond's watershed. Stoughton at this time did not own all the immediate watershed

of the pond. In the preceding three years, about 69 acres of land had been sold to Stoughton, giving the town complete control over Muddy Pond and vicinity. The problem of clearing this land of dead trees and partially burnt stubs was accomplished with the aid of the Work Projects Administration. The W.P.A. cleared about 30 acres on the west side of the pond and planted it to pine seedlings. These pines are now two to three feet in height and doing nicely. This reforestation program also provided for the building of roads and firebreaks through the newly planted area. Thirty thousand pines were planted, and several miles of roads and firebreaks were completed. In the winter of 1940 to 1941, the W.P.A. is working on about 30 acres of the east side of the pond. Scrub trees and brush are being cleared, and the best trees are being left to grow more freely in this area. A gravel road is being built, and several fire breaks will be made criss-crossing the reservation. It has been planned to set the section to pines in the early spring of this year. The slow growth of these pines, however, will consume many years before the trees will be of any great advantage in protecting the watershed of Muddy Pond. Between 1935 and 1937 the quality of the water in the pond grew steadily worse. The taste and odor of the water increased so that they were even noticeable in the more normal times of the year. By normal times, I mean times when there has been no heavy downpour of water, no fire, nor heavy drag on the reservoir. The color of the water, from the State Department of Health's periodic analyses, increased from 18 p.p.m. in 1936 to 120 p.p.m. in 1938 during the worst season of the year, usually in the month of October. The results of the dry weather of the late summer and early fall usually show up in about a month, or just before the regular prewinter rains. An investigation of the situation was started, and the cause of trouble was soon discovered. In going back through the annual pumping-station records, the yearly consumption of water showed a gradual increase from a daily average of 477,000 gal. in 1928 to 734,000 gal. in 1935, then a corresponding decrease in consumption to 564,000 gal. in 1937. This meant that, in the later years, most of the water remained in the pond for a longer period of time. The water that came from the springs at a temperature of 45° F. flowed into the pond and became, in the summer, as warm as 72° F. on the surface of Muddy Pond, the pond being only 3 to 4 ft. deep. Previously, soundings had been made in and around the reser-

voir. The bottom of the pond was shown to contain over 40 ft. of mud gradually decreasing to approximately 10 ft., 100 ft. from the shore and in a section of the channel leading from the pond. Muddy Pond, therefore, was appropriately named, for it was a mud bowl. The examination of the pond itself showed that there were considerable growths of algae and weeds and that the surface, at times, was overgrown with floating masses of organisms that would be dislodged by the wind and due to heavy draft would then flow through the outlet channel to the screens at the entrance to the 15-inch pipe inlet. It was impossible to keep the outlet channel clear without stirring up muddy water. The decrease in consumption and corresponding increase in detention of water in the supply together with the wide range in temperature from 45 to 72° F., or an increase of 27° F., rendered the condition of Muddy Pond worse. The channel became plugged with floating masses of organic matter, and all the water used by the town was being strained through this medium before it entered the pumping system.

Ground Water Collecting Works. The State Department of Health was asked to make analyses of water in the pond and from the springs in the hills. Samples from the pond showed, as usual, nothing that was likely to harm any one, and the samples from the springs showed water of excellent quality. The pond itself was considered too small and shallow to be treated easily for taste, color, and odor control. The State Department of Health recommended that Stoughton give consideration to ground water collecting-works in the vicinity of Muddy Pond or to treatment of the water by some means of filtration. The former suggestion of a ground water collecting-works brought to mind the recommendation repeatedly suggested by consulting engineers. When anything was to be done about expanding or improving the water supply, the first suggestion made was to intercept the water at its source. The cost of such projects, however, was prohibitive and the alternate suggestions made would be the by-passing of polluted brooks or the digging of the outlet channel to a wider or deeper section.

In 1938, it was decided to examine the possibility of laying a gravity line around the bond against the foothills where the springs flowed into Muddy Pond. It was decided that the job could be done, and a W.P.A. project was compiled to accomplish this work. The

project was approved by the State Department of Health and by Washington, the work starting in December 1938. A tight-joint 24-in. reinforced-concrete pipe was to be laid, from below the puddle dam connecting with the 12-in. gravity line to the pumping station, to a point 300 ft. south and west of Muddy Pond. At this point a screen chamber was to be built. The purpose of this chamber was to act as an overflow to the pond, to control the water level in the ground. The 24-in. concrete pipe was not only tongued and grooved but it also had a 6-in. reinforced collar for added strength and was brought in 4- and 8-ft. lengths. The entire 1400 ft. were laid through Fuller's Earth at a maximum of 14 ft. and a minimum of 10 ft. in excavation. It required eleven months to complete this part of the job in this difficult digging. An average of less than 15 ft. of pipe was laid to grade per day. The foothills of the channel section of the basin were saturated with water and would develop a terriffic push, and double sheathing and 6-in. by 6-in. hard pine whalers were used in construction. After several different methods had been tried, most of the excavation was accomplished by hydraulic methods. Water was pumped into the trench stirring up the earth which was then pumped out and deposited on the bank. Working by any other method seemed to produce a quicksand condition, and when a shovel of dirt was removed more Fuller's Earth took its place at the original level. The pipe was laid on a bed of gravel and stone, and compound joints were poured to seal the pipe against leakage. The 24-in. concrete pipe was finally placed, and luckily nobody was injured in the construction. On the upstream or southerly side of the screen chamber, a 24-in. double-strength tile line 826 ft. long was laid around the spring-bearing foothills. The first 200 ft. were laid with calked joints, and the trench was backfilled with stone and material excavated. There was not a great deal of water in the surrounding excavation, although at this point the new pipe line was within 150 ft. west of the pond. This showed how impervious was the mud basin that contained the pond. From this point on, the ground was entirely water-bearing gravel. This gravel was the most ideal filter possible, as the water samples from the springs had shown. The pipe from this point on was to be laid with open joints. The velocity of approach of the spring water was going to be great enough to carry sand into the open joints, and in a short time the main pipe line would have filled with sand. A

filter medium was, therefore, necessary around the pipe. Test pits showed an excellent gravel bank on the property, and a crew of W.P.A. men screened gravel for a filter. On the surface of the water-bearing gravel, against the foothills, was a luxuriant growth of trees and weeds with an average of one foot of loam above the gravel bed. The trees were cut, and a gasoline shovel was hired to remove the stumps and the loam which was full of roots and costly to remove by hand labor. This shovel cleared a strip of ground, 30 ft. wide, back of the pond and against the foothills, the dirt being piled towards the pond, making a dam. The object of this dam was to prevent the pond, if it should ever overflow its mud basin, from seeping back into the new system, for the flow line of this new ground water collecting-system lies 4 ft. below the pond level. Three sizes of stone and fine sand were screened out of the gravel bank: pea stone or ¼-in. stone, ¼- to ¾-in., ¾- to 1½-in., 1½- to 3-in. stone, and fine sand. The bottom and sides of the trench were lined with 3 to 6 in. of pea stone to stop the running of sand. Around the bells of the pipe, 1½- to 3-in. stones were placed to stop the smaller stones from entering the pipe. Above the pipe were laid 1½- to 3-in. stones and successive grades of stone down to fine filter sand at ground level. The purpose of the filter medium above the pipe was to cleanse the rapid run-off of surface water from the hills of foreign matter before reaching the pipe and also to act as a reservoir during idle periods and slowly to increase the velocity of approach when the drag on the supply starts. From the main open-jointed line, 12-in. tile laterals were laid against the very toe of the hillsides in order to intensify collection capacity. These trenches were also back-filled with filter material. In the process of laying and excavating this gallery, some sections were found to carry more water than others and 12-in. lateral tile pipe was laid into these points. Deep wells were created with a filter medium placed around them, and the water was so led more easily into the main pipe line. From the end of the 24-in. tile line, 275 feet of 18-in. tile with 12-in. laterals were laid and backfilled in the same manner. Toward the completion of the work it was necessary to pump at a rate of 1500 g.p.m. in order to lower the water table so that the men could work.

In this water-bearing section, at the head of Muddy Pond, were two steep and short valleys or ravines in the hills. Water commenced

to seep out of these valleys at a level 20 ft. above the normal surface of the ground where the filter gallery was being laid. Two streams of about 100 g.p.m. showed a tendency to wear a brook across the new filter bed. Therefore 12-in. laterals were laid in these valleys and covered with 3 to 4 ft. of filter material. This formed an added supply. During the construction, Muddy Pond was supplied with water from these two valleys and springs on the south-easterly side of the pond.

When all the pipe had been laid and the trenches had been back-filled with filter material, the surface of the strip of ground 30 ft. wide that had been grubbed off with the gasoline shovel was back-filled with fine filter sand as an added precaution for filtration. The job was completed in 20 months. The pipe line started below the old puddle dam, passed around the pond outlet channel and under the apparently polluted Drake stream to a screen chamber. This chamber contained an outlet which controlled the level of the spring water in the pipe filter gallery, keeping it below the ground and out of the sunlight. From the screen chamber, the pipe extended in a half-moon around the pond against the foothills, intercepting the ground water in a filter gallery before it could pass on its way into the pond and be exposed to warmth, sunlight, and surface vegetation.

This system was put into service on July 10, 1940, after due inspection and analyses by the State Department of Health. The results were splendid, and consumption increased 45%, but we do not flatter ourselves with this figure, for industrial business increased somewhat throughout the town. With this heavy demand, our pumping station was working 24 hours a day. In the last days of the month of August the ground water table began to show a gradual drop at the source of supply. There was no immediate danger of scarcity of water, but it was well to analyse the cause. Statistics showed us that we were short 6 in. in our total rainfall for the year 1939. During the construction of our filter gallery, Stoughton was supplied with water from Muddy Pond. In order to construct this filter we had to pump the ground water down to the flow line of our proposed main pipe lines. This level was 4 ft. below the pond level that lay in the mud bowl. Therefore the pond, relying on the same source or ground water table as our construction, fed Stoughton while we were pumping to waste from our excavation. This produced about three times

the normal drag on the adjacent ground water table and extended over a period of 20 months through a dry period. It was natural, therefore, that in the dry months of August and September we should find a lowered ground water table. As a precautionary measure and to prevent using the storage of the discarded pond, it was decided to dig two wells at the end of our 18-in. trunk filter gallery line. These wells were sheathed 8 ft. wide on the surface and telescoped to 5 ft. width at the 12-ft. level. All went well until the digging reached the 12-ft. level and a hard crust was encountered. On breaking through this crust so much water appeared, that the pumps could not take care of it. The pumps were rated at 640 and 350 g.p.m. respectively and were only two or three years old. It was agreed that they were probably discharging about 500 g.p.m. toward Muddy Pond for the water was coming with so much force that it stirred sand up in the wells. The same condition was encountered in each well. While we were pumping on these wells, we experimented to see if the sand could be removed from the bottom of the well and replaced with pea stone. Soundings were taken and it was found that there were about 35 ft. of water-bearing sand and gravel in this area. Also the water level in the filter gallery was recorded, and no indication or effect of this heavy pumping could be noticed. This experimenting or pumping lasted ten weeks, and for eight of these weeks pumping was continuous night and day. These wells were allowed to drain until the sand stopped boiling to the surface. Then the wells were piped into the 18-in. tile collecting gallery. It is planned to dig three or four more wells in the spring of 1941 in order to prove further both the reserve capacity of this new gallery system and to provide for increased consumption should the ground water table not be replenished. We should remember, in this connection, that this system cannot lower the ground water table because our screen chamber control waste outlet is 5 in. higher than the original Muddy Pond supply level.

In summing up the work done: 3500 ft. of 24-in., 18-in., and 12-in. pipe were laid with the necessary manholes and a screen chamber. Excavation amounted to 9,786 cu. yd. and, where filter material was needed around one half of the pipe laid, this yardage had to be dug out of a gravel bank, screened, transported and placed around the pipe and wells in the filter gallery. This work was carried out through the winter months regardless of frost, in order to give the W.P.A. men their working hours. The town's contribution was about

$27,000 and the federal contribution about $59,930. The town used its money for materials and machines such as the gas shovel, water pumps and pipe. It is safe to say that with increased consumption and dry weather the town of Stoughton would have been dangerously near a water shortage in the fall of 1940, if the new system had not been installed. Also, the town of Stoughton would never have attempted this filter gallery system if the expense had had to be borne without a contribution from the government in the form of W.P.A. funds. The increasing demand for water has caused the pumping station to average better than 20 hours of running per day. The maximum month was October, 1940, averaging 740,000 g.p.d. against normal averages of 505,000 and 547,000 g.p.d. in 1938 and 1939 respectively. The 12-in. cast-iron gravity main from below the puddle dam at Muddy Pond will not deliver much more water on the grade on which it is laid. This bottle neck of approximately 4300 ft. needs a little investigation, therefore. It is hoped that, in the near future, with the help of W.P.A. funds, Stoughton will be able to parallel this 12-in. main with another pipe, 12 in. in diameter or larger, in order to bring more water to the pumps 4300 ft. away from the source of supply. The supply is estimated to be capable of delivering 1½ m.g.d. during the dryest periods. This is more than double our normal consumption. Stoughton not only has developed a good source of water supply but has also remodeled the town's distribution system. There are roughly 55 miles of town highway and 48.5 miles of water mains, with 464 hydrants. From the regular analyses of the new supply, the water is found to contain more CO_2 than the old system and this will be taken care of by treating the water with lime or soda ash. The question of continuing the application of chlorine was put before the State Department of Health. A W.P.A. crew is clearing and grubbing the east side of the water supply reservation, and we plan to enlarge the gravity trunk line between the source of supply and the pumping station. When all the work is done, and a reasonable time has elapsed, use of the chlorinating apparatus will be discontinued.

Stoughton has a water system which is paying interest on its investments and would be a credit to any community. We believe that the experience of the community during the last few years will impress the citizens with the present quality of the water, and that popular demand will keep us on our toes to maintain one of the best water departments in the state.

POLLUTION OF THE ROCHESTER, N. Y., WATER SUPPLY THROUGH CROSS-CONNECTIONS

BY EARL DEVENDORF*

[*Read February 20, 1941.*]

One of the most serious emergencies to confront the New York State Department of Health in recent years in connection with the pollution of a public water supply occurred at Rochester, N. Y., on December 11, 1940. At this time about 75% of the 500-mile distribution system of the city became contaminated by raw Genessee River water pumped accidentally through an unknown cross-connection between the fire supply (Holly System) and the municipally owned potable public water supply (Hemlock System).

Water Supply Systems of the City. The city of Rochester, having a population of some 325,000, is served by two potable public water supplies, the one, municipally owned, supplying approximately 80% of the population and the other, owned by the Rochester and Lake Ontario Water Company, furnishing water to about 20% of the population of the city, as well as an area outside of and adjacent to the city proper. In addition, there is a separate municipally-owned high-pressure, fire-fighting system serving the business and industrial districts of the city.

The municipally owned system, known as the Hemlock Supply, is obtained by gravity through three conduits from Hemlock and Canadice lakes, located some 35 miles south of the city. This supply is chlorinated before delivery through Rush Reservoir, situated some five miles south of the city, and two distributing reservoirs in the city (Cobbs Hill and Highland Park reservoirs).

The supply of the Rochester and Lake Ontario Water Company is obtained by pumping water from Lake Ontario and filtering and chlorinating it before delivery. The company supply has no connection with the Holly fire system and was not affected by the recent pollution episode.

Relation of Fire System to Potable System. The fire supply is

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a separate system installed in 1873 and consists of about 25 miles of mains supplying hydrants and automatic sprinkler equipment in the congested mercantile and industrial area of the city. Normally the pressure in the fire-supply system is maintained by either or both of two 500-g.p.m. pumps that draw water from a small suction well located at the fire-supply pumping station. This well is supplied only with water from the potable system. In addition, four 2100-g.p.m. fire pumps take suction from a large well at the pumping station which may be supplied either with river water or through a 12-in. main with water from the potable supply. This main discharges into the suction well above the flow line. Under normal conditions, the fire pumps are supplied from the potable supply but, in case of large fires, the valve controlling the river-water supply is opened, and river water is permitted to enter the suction well. The normal pressure on the fire system is 95 lb. per sq. in., which is raised to about 125 lb. when the fire pumps are operating.

Although original construction records of this system are no longer available, the plans on file show that there were, at one time or another, some 13 or more cross-connections between the fire system and the potable water supply system. In 1928, the city manager of Rochester ordered the elimination of the ten cross-connections known to exist at that time between these systems. The removal of these cross-connections was reported to have been carried out. Record maps of the city water department indicate, however, that the valves on these connections were "sealed" rather than removed. In at least one instance, the valve had been "sealed" merely by filling the gate boxes with broken stones.

Occurrence of Pollution. On December 11, 1940, following a reported leakage from a valve on State Street, a crew of the Water Department was assigned to make repairs. It was necessary to excavate in order to reach the valve and, on completion of the repairs, a workman, having found the valve closed, opened it without notifying or checking with the Water Department officials. The workman acted apparently in the belief that the valve was on the potable-water system instead of on one of the old cross-connections between the fire and the drinking-water system.

Immediately after this valve was opened, the automatic alarm in the pumping station began to ring and the gauge in the pumping

station showed a marked reduction of pressure in the fire system the pressure in which is normally much greater than that in the potable-water supply system. When it was found impossible to maintain pressure with the small pumps, the large fire pumps were put into operation in an effort to re-establish normal pressure in the fire system. During the first hour, the fire pumps were supplied with Hemlock potable water discharged into the suction well through the overhead supply pipe. When it became apparent that the capacity of this pipe was inadequate to furnish sufficient water for the fire pumps, the valve on the Hemlock supply line was closed and the valve permitting river water to enter the suction well was opened. As a result, the highly turbid river water was pumped into the fire system and, through the open valve on the cross-connection, into the Hemlock potable-water supply distribution system. This pumping of river water continued from about five o'clock in the afternoon until about eight o'clock in the following morning, December 12, 1940.

In the meantime, the inability to maintain adequate normal fire pressure in the Holly system had been reported to the Water Department, and emergency crews had been dispatched to search for the supposed leak in the fire system. This search was continued until some time after midnight. It was not until the following morning, December 12, when the workman who had repaired the valve on the previous day reported the opening of the valve, that the cause of the supposed leak in the fire system became known. Hereupon the valve was closed.

From records of the Water Department and from the rating curves of the fire pumps, the volume of river water pumped into the drinking-water supply system during the intervening period was found to be approximately five million gallons. The pumping of this large volume of river water resulted in widespread pollution of the domestic distribution system, particularly since the river water was pumped during the night when consumption of water in the city was at a minimum.

Procedure Taken to Notify Public and Remove Pollution. As soon as this situation became known, Dr. A. M. Johnson, Health Officer of the city, immediately notified the State Department of Health and initiated steps to warn the citizens of the serious situation and to advise them by radio broadcast, newspaper articles, and tele-

phone to boil all water before drinking it. All public buildings were notified, and 220 special telephone operators communicated with 95,000 subscribers advising them to boil their drinking water. "Boil water signs" were posted in buses, stations, traffic zones and street cars. Guests who had been registered in hotels on December 11 and 12, 1940, were advised by letter of the pollution of the drinking-water supply and urged to become innoculated against typhoid fever. All physicians were asked to be on the alert for gastroenteritis and to report cases promptly to the City Health Department. Dr. Johnson organized typhoid immunization clinics in the Rochester Health Bureau and in the out-patient departments of hospitals. Additional clinics were organized in East Rochester and the adjacent town of Greece.

Arrangements were immediately made for daily laboratory examinations of samples of the water from some 50 sampling stations established throughout the distribution system for the purpose of determining (1) the extent of spread of pollution through the city mains and (2) the time when pollution had been removed and the use of unboiled water could safely be resumed.

As soon as the situation became known the City Water Department arranged for flushing the system and installing chlorine apparatus to treat the water flowing from the Cobbs Hill and Highland Park reservoirs in an endeavor to sterilize the distribution system and to eliminate as much of the polluted water as could be removed by widespread flushing of hydrants.

Chlorination of Potable Water Supply with Hypochlorite. Following a conference of local and state health officials on the evening of December 12, immediate steps were taken to obtain additional equipment in an endeavor to increase the amount of chlorine applied to the water supply from Cobb's Hill and Highland Park reservoirs. Chlorine equipment was obtained from adjacent municipalities and from manufacturers in Buffalo, but it was apparent, after the second day, that the chlorine residual was increasing only very slowly and was not reaching all portions of the distribution system. Accordingly, on the morning of December 15, arrangements were made to procure a ton of HTH (high-test hypochlorite). With the assistance of personnel and equipment from the Department of Public Works, including street flushers and pumpers, hypochlorite solution was pumped into the water distribution system through hydrants. Six street flushers

having a capacity of 2500 gal., each with two fire pumpers, were used. Routes were mapped out covering the whole north end of the city, both east and west, and a short distance in the southerly section where preliminary results of laboratory examinations indicated that the water supply system had been polluted.

The street flushers were equipped with pumps that facilitated recirculation and mixing of the solution. Five pounds of HTH were added to about a foot of water in each flusher tank and recirculated for a period of about five minutes. The flusher was then filled with water from a hydrant and the pumper hooked up, taking suction from the flusher and discharging the hypochlorite solution into a hydrant against the city pressure. By this method, a solution containing approximately 165 p.p.m. of available chlorine was pumped into the distribution system. A hydrant on the main under treatment was opened at a place 400 or 500 ft. from the point of application, inducing a flow estimated at about 250 g.p.m. at the point where the chlorine solution was being introduced and resulting in a rate of application of chlorine of about 5 p.p.m. A large number of field kits for making orthotolidine tests was provided for the workmen. Immediately upon obtaining a chlorine residual, the hydrant was closed and a hydrant further from the point of pumping was opened until a sample also indicated the presence of chlorine residual. At the end of the first night's pumping, chlorine residuals in samples collected throughout the distribution system indicated that the residuals were being increased at the points desired.

Pumping of the hypochlorite solutions was repeated during the night of December 16. Tests made throughout the distribution system during the following day indicated that the residual chlorine content throughout the system was approximately 0.4 to 1 p.p.m., except at dead ends. Throughout the day of December 16, water works employees were assigned to flush some 165 hydrants on dead ends until a chlorine residual was obtained.

On the evening of December 17, at a meeting of city and state health officials, it was agreed, on the evidence of laboratory tests and orthotolidine tests of samples collected from the distribution system from many points, that the system had been effectively chlorinated and that the water supply was safe. Accordingly, Dr. Johnson announced to the public that boiling the city water was no longer necessary as a safety measure.

It was indeed fortunate that the Department of Public Works of the city of Rochester had available a large number of modern street flushers and pumping equipment, together with an efficient and well organized force. This equipment and the available organization made it possible to accomplish the sterilization of the large mileage of water mains in a fraction of the time that would have been required had sterilization of the distribution system been effected simply by the introduction of chlorine into the water supply reservoirs. So far as known, this method of sterilization of a large distribution system has never been employed heretofore. It proved both successful and effective.

Effect of Pollution. Based on a survey made by Dr. Johnson, it is estimated that some 30,000 cases of mild gastroenteritis occurred among the city residents following the pollution of the public water supply. However, only five cases of typhoid fever were reported during this period. Some 28 individual suits for damages in connection with the occurrence have been filed against the city with total claims amounting to about $125,000.

There were a number of factors that tended to prevent the development of an extensive epidemic in spite of the excessive pollution indicated by bacteriological examinations of samples of water throughout the city and the initial wave of gastroenteritis.

1. The city of Rochester no longer discharges its own sewage into the Genesee River and all but one of the municipalities above Rochester have sewage-treatment plants.

2. Because of rains and melting snow, the river flow was high and pollution present was greatly diluted.

3. Typhoid fever is not nearly so prevalent as formerly so that sewage contains fewer typhoid germs from active and convalescent typhoid cases. In addition the number of typhoid carriers undoubtedly is constantly diminishing. Only a few years ago, an initial wave of gastroenteritis following such heavy pollution of the water supply would inevitably have been followed by a wave of typhoid fever.

4. The time elapsing before the institution of corrective measures and warnings to the public that the water should be boiled was comparatively brief. The methods used for spreading the knowledge of pollution were probably more extensive than any heretofore employed. In the past, discovery of existing pollution has frequently

been delayed until the outbreak of gastroenteritis suggested its presence, and water has been polluted for a matter of days rather than hours before remedies were instituted.

Plans for Abandonment of River as Source of Fire Supply. It was quickly realized that continuance of the present system of fire supply raised uncertainties as to the control of future occurrences such as this because there are no early construction records of the fire pressure system, which was installed in 1873. An unknown cross-connection between the fire pressure system and the Hemlock domestic system was discovered, for example, in the summer of 1940 during a paving project. The availability of the Holly fire pressure system in a large number of industrial and commercial plants, furthermore, presents the possibility of new cross-connections being made by an employee without his realizing the dangers therefrom. Therefore, in view of the above and the uncertainty of the possible existence and location of other cross-connections, it was felt that the continued use of river water as an auxiliary supply would be an ever present hazard to the safety and health of the citizens of the city.

The author, accordingly, recommended to the city administration that immediate steps be taken to provide for the abandonment of the use of river water as a source of auxiliary fire supply and arranged a conference between representatives of the city and engineers of various fire insurance interests to consider this problem. The fire insurance engineers were asked to study the problem with a view to formulating a plan, to be submitted to the city, that would provide for the abandonment of the use of river water and for increasing the capacity of the Hemlock water main to the present fire pumping station with possible increased storage facilities or a combination of both.

At a subsequent conference, held in New York City in the office of the Chief Engineer of the National Board of Fire Underwriters, between representatives of the city administration, the state department of health, and engineers for the various fire insurance interests, a plan for abandoning the use of river water was tentatively agreed upon. This plan provides for the installation of a 3-m.g. covered storage reservoir, supplied only with water from the potable water supply system from which the fire pumps will then draw water exclusively for fire-fighting purposes. William H. Roberts, Commissioner of Public Works of Rochester, and his engineers are at present pre-

paring plans for the carrying out of this recommended change. Richard H. Ellis of the Associated Factory Mutual Fire Insurance Companies, who has followed this problem, will outline the details of this problem and the recommendations made for changes from the fire insurance standpoint.

As a result of this occurrence, Dr. Edward S. Godfrey, Jr., State Commissioner of Health, sent the following letter to all municipal water supply authorities in the state:

"The recent experience in the city of Rochester where a polluted municipal fire supply was accidentally pumped into the city distribution system through a cross-connection, which resulted in a large number of cases of illness among the residents of the city and gave rise to claims for damages, again indicates the danger from illegal cross-connections between potable and non-potable water supplies.

"This Department in 1926 advised all those in charge of public water supplies of the enactment of the State Sanitary Code, which prohibits the maintenance of cross-connections between potable public water supplies and non-potable supplies and places the responsibility for the elimination of all such cross-connections upon the local water supply authorities.

"As the result of that notice we were advised that all known illegal cross-connections between potable and non-potable water supplies were eliminated. This was accomplished (1) by completely severing many cross-connections, or (2) by the discharge of potable water into pump suction pits or elevated tanks above the flow line of these structures, which pits or tanks could be filled also by non-potable water, or (3) by the installation of fire pump chlorinators together with double check valves of the all-bronze type.

"I would request that an inspection be made of all factories, mills, mercantile establishments, office buildings, theatres, and other properties in your community that may maintain auxiliary water supplies for fire protection, industrial use, air conditioning, etc., and that immediate steps be taken to eliminate or protect all illegal cross-connections found to exist. This inspection should include also an examination of permissible cross-connections provided with fire pump chlorinators and double check valves of the all-bronze type.

"Please report to me at your earliest convenience the results of your inspection, including:

"(1) The number and location of all illegal cross-connections discovered.

"(2) The action taken to require the elimination or correction of illegal cross-connections.

"(3) When the correction or elimination of such cross-connection will be ready for inspection by our engineers.

"(4) Action taken to see that no illegal cross-connections are installed in any industrial plant building or other structure that may be built in your municipality.

"We shall be glad to cooperate with you in eliminating illegal cross-connections, so please do not hesitate to call upon us should you need any assistance."

As a result, an intensive campaign of investigation is now being carried out by all water works authorities in New York State. Many illegal connections that apparently were unknown have been discovered already, and steps are being actively taken by the District Engineers of the Department to follow up this survey with a view to having all such known cross-connections eliminated.

The Rochester incident, as well as the results thus far obtained in the campaign of investigation now under way, illustrate clearly the necessity for diligent search by water works authorities to discover and eliminate illegal and dangerous cross-connections.

Historically, it is of interest to learn of a previous episode of pollution of the Rochester drinking-water supply system through a leaking check valve. This occurred on April 22, 1898. The information was received from Edwin A. Fisher, Consulting Engineer Emeritus of the City of Rochester, who obtained the same from the private notes of Emil Kuichling, former City Engineer and also a prominent consulting engineer who acted in an advisory capacity for the State Department of Health. The following is a transcription of Mr. Kuichling's notes regarding this previous episode:

"In searching for cause of roily water in 1st and 2nd wards, in response to complaints, Mr. Hotchkin finds that the check-valve between the Holly and Hemlock systems of Fire Sprinkler supply to the Atwater, Armstrong and Clarke building, corner of Allen and Platt streets, had become defective and was leaking badly, allowing the river water to be forced into the mains of the Hemlock system at that point.

"I at once ordered the Hemlock sprinkler Service to be shut off, and to remain shut off permanently. Gave notice of this action verbally and in writing to representatives of the owners.

"Advised both parties to arrange with their respective Fire Insurance agents with respect to their fire insurance.

"I also directed Mr. Hotchkin to flush the Hemlock mains in 1st and 2nd wards by districts."

Discussion

Richard H. Ellis.* The City of Rochester's direct-pumping fire system, known locally as the Holly System, was originally constructed in 1873 to supply high-pressure hose-streams directly from hydrants within the principal business and industrial areas. The

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system was designed solely for fire service and supplemented the supply available at low pressure from the potable Hemlock System. At that time, the pressure in this latter system fluctuated considerably. In 1895, Emil Kuichling of the Rochester Water Works stated that the pressure fluctuated from between 10 and 40 lb. per sq. in. The Holly distribution network, which was laid throughout the congested area, was expanded so as to consist ultimately of 25.65 miles of 4, 6, 8, 10, 12, 16, 20 and 24 in. pipe.

As the name implies there is no storage provided in the Holly System. Genesee River water was continuously pumped to the grid-iron system from Brown's Race by two, 2-m.g.d. water-wheel-driven pumps and by two steam-driven pumps, each of which had a capacity of 3 m.g.d., thus providing a total plant capacity of approximately 7000 g.p.m. This station was electrified in 1925, at which time two 420-g.p.m. and four 2100-g.p.m. pumps were installed. Later the 420-g.p.m. pumps were replaced by the present 500-g.p.m. units. As described by Mr. Devendorf, one or both of the small pumps are ordinarily operated to supply a limited industrial consumption and sufficient make-up water to offset such leakage as would be expected from a distribution system of this size.

The pumping equipment at the Brown's Race Station is so arranged that when the capacity of a single 500-g.p.m. pump is exceeded, the second small unit is automatically started. Should the capacity of both these units be exceeded, the resultant drop in pressure causes a low-pressure alarm to ring and one or more of the four 2100-g.p.m. units is manually started. The small units ordinarily maintain from 75 to 105 lb. per sq. in. pressure at the pumping station, while the larger units are designed to deliver their rated capacity at 145 lb. pressure per sq. in.

To provide both increased reliability and capacity, a second Holly pumping station was constructed in 1906 on Johnson's Race. The pumping equipment there located consisted of a single 2100-g.p.m. electrically-driven pump which was supplied with Genesee River water from Johnson's Race. This made immediately available a limited quantity of water at such times as it was necessary to close Brown's Race for cleaning or for the repair of gates etc. This station was abandoned in 1939, presumably to effect economy of operation.

The continual pumping of river water resulted in the deposition

of an appreciable amount of silt in the Holly mains, the carrying capacity of which was so impaired that considerable mechanical pipe cleaning was undertaken between 1908 and 1918. In order to prevent further silting of the underground pipe system, the two small base-load pumps were arranged, in 1925, to draw suction from a 2000-gal. capacity well which was filled automatically through a float-controlled valve from the Hemlock System. The four large pumps continued to take suction from a 32,000-gal. well filled by gravity with river water. A 12-in. connection from the potable water system was arranged to discharge, above water level, to this well and this supply, estimated at 5000 g.p.m. was also available in the event that the Race was shut off for repairs.

In opening this discussion the writer has referred to the Holly System as having been originally designed to supply high-pressure hydrant-streams. With the introduction of the automatic sprinkler in 1875 and with the rapid acceptance of this type of protection between 1880 and 1890, the Holly System became an important source for private fire-protection installations in the higher buildings, as the pressure available from the Hemlock System was inadequate for such protection. Despite improvements to the Hemlock distribution mains which resulted in increasing the pressures to an average of from 40 to 55 lb. in the business and industrial district, the practice of utilizing the Holly System to supply automatic sprinkler installations in high buildings has been continued. Buildings of four stories or less are frequently supplied from the Hemlock System. It is now conservatively estimated that there are 150 sprinklered properties, having a value of at least $50,000,000 with water supplied either in whole or in part from the Holly System. There are a number of such properties where gravity tanks furnish a limited secondary supply. Within the district supplied by the Holly System, a credit of 5% on the base rate for unsprinklered properties is established by the New York Fire Insurance Rating Organization. While no estimate has been made of the value of the unsprinklered properties so benefited, it is evident that, beside making available desired high pressure for the protection of sprinklered properties, a definite economic advantage accrues to the owners of unsprinklered buildings from the maintenance of this system.

For a few months prior to the occurrence of the cross-connection

difficulty described in Mr. Devendorf's paper, the fire insurance interests had been investigating the reliability of the Holly System and had made such recommendations to the city authorities as would satisfactorily increase the reliability of that system. These recommendations included some changes in the automatic control of the small pumping units, the provision of a second pumping station to have a suction supply that would be independent of that available to the pumps at the Brown's Race Station and improvement in the carrying capacity of the distribution system. Until the occurrence of the unfortunate event on December 11, 1940, the insurance organizations, so far as the writer can determine, had no knowledge that the potable and non-potable systems were physically interconnected. Most definitely the writer's own organization understood that there was no physical connection between the two systems.

Following the opening of the valve on this cross-connection, the city authorities, at Mr. Devendorf's suggestion, requested advice from the underwriters as to what changes might be made to make the Holly System satisfactory both from the viewpoint of the health authorities and fire protection organizations. This lead to the conference in New York City at which representatives of all interested parties were present. There it was unanimously agreed, in view of the now known presence of several cross-connections between the potable and non-potable systems, and because of the possibility of the existence of other connections of which there is now no record, that the only absolutely safe procedure is to supply the Holly pumps with potable water.

The total fire demand for a city of this type having a population of 325,000 would be established, according to the National Board of Fire Underwriters Schedule, at 16,000 g.p.m. While it would be possible at some places in the congested areas to supply nearly this quantity from the Hemlock System, the pressures throughout the business and industrial areas would be so lowered thereby that the maintenance of automatic sprinkler protection in those properties supplied from the Hemlock System would be seriously impaired. To supply the entire demand of all the Holly pumps (approximately 10,000 g.p.m.) directly from the distribution mains in the Hemlock system would result in lowering the Hemlock pressure to from 20 to 25 lb. in most places. It was, therefore, felt important, if Hemlock water

was to be made available for suction supply of all Holly pumps, that (when supplying the maximum plant capacity) at least a portion of the supply should be drawn from storage. Recommendation was, therefore, made for the erection of a 3-m.g. aboveground steel suction tank to be located near the Brown's Race Station. A tank of this size, connected to the Hemlock system so that a controlled flow of about 5000 g.p.m. would be at all times available thereto, could furnish the total suction demand of approximately 10,000 g.p.m. for a period of ten hours without drawing the pressure in the Hemlock system below approximately 45 lb. Furthermore, in the event of an extreme emergency requiring the full 16,000-g.p.m. demand, this quantity could be made available while maintaining a residual pressure of from 20 to 25 lb. in the Hemlock system.

Additional recommendations were made for the proper cleaning and sterilization of the pipes in the Holly distribution system and for the provision of steam or Diesel-engine drive for two of the 2100-g.p.m. pumps and for several other improvements designed to increase the reliability of the system.

The utilization of potable water only in the Holly System will eliminate the possibility of a recurrence of the December 1940 difficulty and will in that way satisfy the health authorities. The provision of suction storage and a second source of power for operation of two of the large pumps will increase the reliability of the system and will make possible the supply of an adequate quantity of water for the protection of the business and industrial districts. Thus, for a reasonable investment by the city, the high-pressure system can be improved and the system made acceptable to the State Health Department.

The opening of the valve on this cross-connection between the potable and non-potable systems in Rochester, N. Y., again emphasizes the hazard attendant upon unprotected cross-connections but contributes little to our knowledge of the hazards involved. It appears that the principal lessons to be learned from this occurrence are: (1) the value and effectiveness of using radio, telephone, newspapers, etc., in such an emergency as this promptly to notify the public of the attendant hazards so that people may present themselves for inoculation and take precautions to boil all water used for human consumption or culinary purposes, and (2) the difficulty of effectively

chlorinating a distribution system from the source and the value of promptly treating various parts of the system with concentrated doses of hypochlorite solution and of following the chlorine residual with orthotolidine tests to ascertain that all parts of the system are reached and properly disinfected.

The City and State Health departments should be complimented on their prompt action in adopting measures to prevent serious consequences such as might logically be expected following the pollution of a large distribution system and on the effectiveness of their efforts thoroughly to disinfect the potable-water distribution system.

SALT-WATER POLLUTION OF THE GLOUCESTER, MASS., DISTRIBUTION SYSTEM

BY FRANCIS H. KINGSBURY*

[*Read February 20, 1941.*]

A cross-connection between pipes carrying water pumped from Gloucester Harbor and pipes carrying water of the public water supply was responsible for the pumping of salt water into the public water distribution system of the city of Gloucester at some time during the night of December 10 to 11, 1940. The first warning of what had occurred was given by a complaint to the Water Department from premises located near the plant in which the cross-connection was later found. Immediately upon receipt of this notification, the Water Department called out its forces, located the cross-connection, and thoroughly flushed all of the public water supply mains in the general vicinity. The Water Department is to be commended for its prompt action in protecting the water consumers of the city.

The Department of Public Health was notified of this occurrence and the situation investigated. Samples of water were collected for bacterial examination and for chemical analysis to determine the chloride content and bacterial condition of the water at various points in the general vicinity of the plant as well as at a point where the water supply mains entered the city. While the results of the bacterial examination of the early samples could not be made immediately available, because of the necessary time of incubation, chemical analyses for chlorides were shortly obtained and showed that the chloride content was normal except in the immediate vicinity of the plant on the cold-water systems. A sample collected from a hot-water system on premises across the street from the plant showed a concentration of 1550 p.p.m. of chlorides, as compared with a chloride content of the harbor water of 17,800 p.p.m.

While it was believed that the thorough flushing of the mains had removed much of the danger to water consumers, it was nevertheless thought wise to notify all citizens to boil all water used for drinking

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and culinary purposes until definite information could be obtained as to the bacteriological quality of the water in the distribution system. Accordingly, the Department advised the local Board of Health and Board of Water Commissioners to thus notify the water consumers, and notices were posted about the city. While all concerned were reluctant to invite the unfavorable publicity, it was recognized that such action must be taken in the interest of protecting the people against the danger of drinking polluted water and to protect the city against possible suit.

Meanwhile, further investigations were being carried on with regard to the details of the cross-connection that caused the trouble. It was found that a direct connection existed between a 2-in. pipe carrying water from the city supply and a 2-in. line carrying water from a pump taking water from the harbor for cooling purposes. A stop cock between these lines and a globe valve on the city water line were the only protections for this cross-connection. Under usual conditions, the stop cock was closed and the globe valve open, supplying city water for the flushing of toilets and for drinking purposes at the plant. Little can be found out as to just what occurred at this plant on the night of December 10 to 11, but it may be assumed that the stop cock must have remained open for a considerable period of time before the presence of salt water in the public distribution system was noticed. Further examination of this plant showed the existence of another cross-connection in a different part of the plant where a 3-in. pipe, carrying water from a deep tubular well and at times from the harbor, was cross-connected with a 3-in. line carrying water from the public supply. This cross-connection was protected only by a single metallic check valve and by a gate valve; but this second cross-connection was not responsible for the pollution of the public water supply.

The results of the bacterial examination of samples of water collected after the first flushing of the mains showed that the water was free from bacteria of the coliform group at all points where the samples were collected, and subsequent samples covering a much wider area about the plant also indicated freedom from bacteria of this type. Although the bacterial examination of water from the harbor showed that bacteria of the coliform group were present in amounts as small as 0.1 c.c., the fact that no disease has been traced to this occurrence may possibly be explained by the circumstance that the city of

Gloucester is provided with a separate system of sewerage and sewage is pumped through an outfall sewer to a point in Gloucester Harbor remote from the city. Upon receipt of the results of the bacterial examination of the final samples collected, it was found that they, too, were free from bacteria characteristic of pollution. Thereupon the Department advised the city that it was no longer necessary for the water consumers to boil the water.

WATER TREATMENT AT GROTON, CONNECTICUT

BY A. N. TIFFANY* and ARTHUR L. SHAW**

[*Read November 21, 1940.*]

For many years in many communities such matters as color, taste, odor, corrosiveness and even pollution of water supplies have been tolerated to the point where they have been taken for granted, or they have been met by temporary or incomplete expedients. More recently, rising esthetic standards have made water users everywhere increasingly sensitive to the quality of water supplies, particularly since the automobile has brought people more and more into contact with conditions in other towns with which to compare their own. Eventually, a public demand for improvement arises which can only be satisfied by permanent and adequate works and methods. This eventuality came to pass in Groton, Conn., culminating about 1938. It resulted in the construction of the water treatment plant that is the subject of this paper.

Source of Supply. The Borough of Groton, Conn., obtains its water supply from Great Brook having a drainage area of about 14 sq. mi. above the Borough Dam which forms a shallow impounding reservoir with a useful capacity of about 125 m.g. Formerly, after heavy chlorination, water was pumped to the distribution system by water power when excess runoff was available at the dam and, at other times, by electric motor or Diesel engine.

The Need for Treatment. The water from the Borough Reservoir was high in color at practically all times, varying in intensity up to 100 p.p.m.; taste and odor were at times troublesome; "red water" difficulties were not infrequent; and analyses of water, sampled at the dam, occasionally indicated pollution to an objectionable degree. Increasing population on and state highway traffic across the watershed and a growing demand for higher standards of clarity and palatability eventually led to the decision, in 1938, to construct a treatment plant and to take advantage of Federal aid in the form of

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a P.W.A. grant. A site was selected on property of the Borough near the existing pumping station and reservoir.

Plant Capacity. Water consumption for several recent years had averaged just under 1 m.g.d., but during the war years it had been between 1.5 and 2 m.g.d. due to the high demand related to intense industrial activity at the shipyards and other plants. A marked tendency toward increase in population, and the possibility of resumption, at least in part, of the industrial demand led to the selection of 2 m.g.d. as the nominal capacity of the proposed treatment plant. Increased activity at the U. S. Navy Submarine Base and at the plant of the Electric Boat Co. in the late summer and fall of 1940 have again changed the character of the demand, and further development may tax the capacity of the system.

Basic Data. Laboratory experiments with water samples taken at various seasons, indicated that the water would readily respond to coagulation and filtration. From these experiments, fundamental data for design were determined.

The general plan of the treatment works is illustrated in Fig. 1, and the following tabulation shows the principal basic data relating to the plant and equipment.

Raw Water Pumps. Three motor-driven centrifugal, 2-700 g.p.m., 1-900 g.p.m.

Aeration. Motor-driven air compressor, variable speed, 16 to 70 c.f.m. free air. Diffuser tubes, 12-3 in. by 24 in. Aeration tank, detention 6.2 min. for 2 m.g.d.

Chemical Feed. Four dry-feed machines, for alum, carbon, pre-lime (or spare) and post-lime. Two manual vacuum solution-feed chlorinators.

Mixing Tank. Detention 17.5 min. for 2 m.g.d. Agitation by variable-speed motor-driven paddle 0.5 to 2 r.p.m.

Coagulation Basins. Two units, detention 4 hr. for 2 m.g.d.

Filters. Four units, nominal capacity of each 0.5 m.g.d. Nominal filter rate 125 m.g.d. per acre. Underdrains, perforated cast-iron pipe. Depth of graded gravel 18 in. Depth of sand 30 in. Depth of water 5 ft. Effective size of filter sand 0.43 m.m. Uniformity coefficient 1.4. Rate controllers 6-in. direct-acting, maximum 0.75 m.g.d. Operating tables 4 units, each with operating levers for 4 hydraulic valves and recording rate-of-flow and loss-of-head gages.

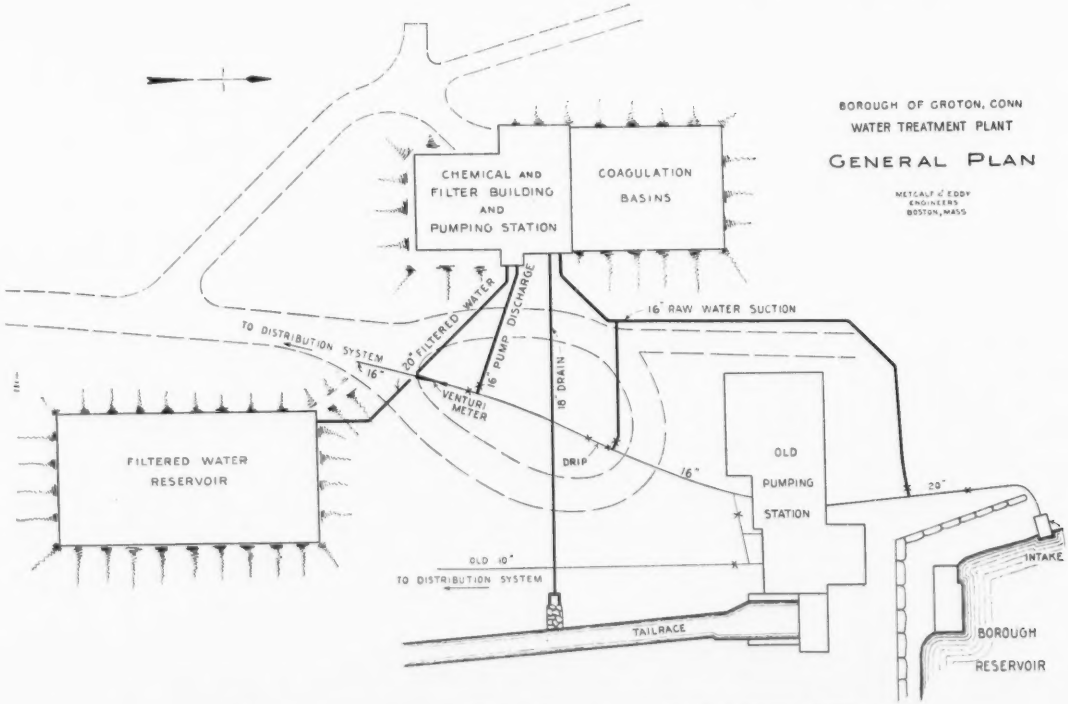


FIG. 1.—GENERAL PLAN OF WATER PURIFICATION PLANT, GROTON, CONN.

Wash Water. Pumps, in duplicate, motor-driven centrifugal, 3,300 g.p.m. Rate controller, 16-in., hydraulic operation.

Filtered Water Reservoir. Covered concrete, capacity 540,000 gal.

High-Lift Pumps. Three motor-driven centrifugal, 1-500 g.p.m. (moved from old pumping station), 1-700 g.p.m. synchronous motor, and 1-900 g.p.m. synchronous motor.

Vacuum Priming System. To keep raw, high-lift, and wash water pumps automatically primed at all times.

Water-Level Regulating Mechanism. Automatically to throttle raw water pumps to maintain water level on filters.

Venturi Meter. Tube 16-in. by 6½-in. located in main high-lift discharge line; register-indicator-recorder on instrument panel in building.

Telemeters. To record water levels in filtered water reservoir, and in distant standpipe, on instrument panel in building.

Electric Power. Incoming power 2200 volts, 3 phase, 60 cycle. Power for main motors, etc. 220 volts, 3 phase, 60 cycle. Power for lighting, etc. 110 volts, single phase, 60 cycle. Transformers, non-inflammable type; power 3-75 k.v.a., delta connected; light, etc. 1-10 k.v.a. Main dead-front switchboard for control and distribution of power and for reduced voltage starters of large motors. Starters for small motors, wall mounted near motors.

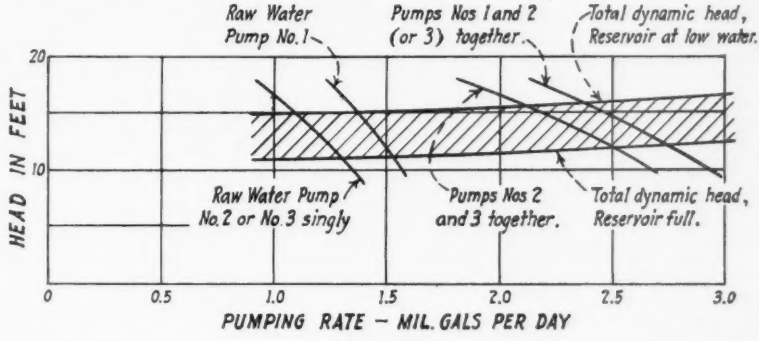
Chemical Storage. At top of building served by electric monorail hoist. Capacity for about 6-month supply of chemicals.

Laboratory. Equipped for routine physical and chemical determinations, and with space for future simple bacterial work.

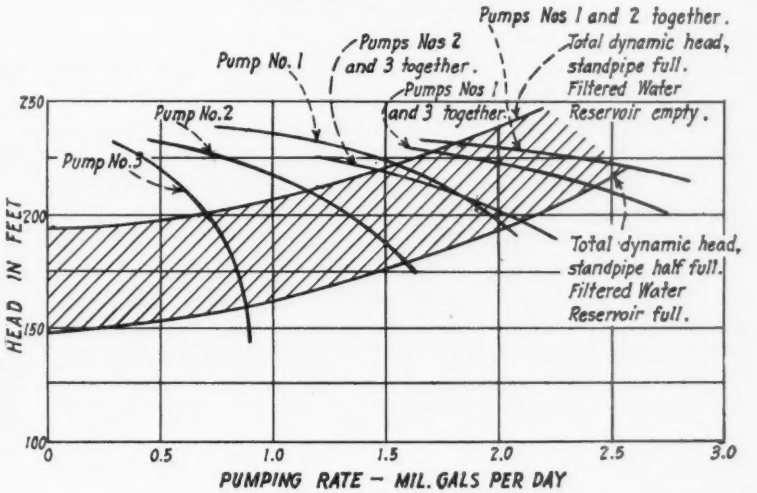
Raw-Water Pumping. Raw water from the nearby Borough Reservoir is carried to the raw-water pumps through a new 16-in. cast-iron pipe connected with an existing 20-in. pipe serving the old pumping station. Water flows from the reservoir into this pipe through an existing intake structure equipped with hand-cleaned screens.

The raw-water pumps together with the high-lift and wash-water pumps and other equipment are located on the ground floor of the new plant. They are Allis-Chalmers centrifugal units, driven by squirrel-cage motors. The pumps are located slightly above reservoir level and therefore must be primed. A Nash Hytor pump automatically exhausts air accumulating in a vacuum tank to which the pump casings are connected, keeping them primed at all times.

On the discharge pipe leading to the aeration tank is an hydraulically-operated throttling-valve actuated by an American Water Softener Co. water-level controlling-device which automatically modulates the inflow rate so as to maintain a constant water level above the filter sand. Accidental overflow due to failure of the water-level control-device is guarded against by an alarm bell which rings if



RAW WATER PUMPS



HIGH LIFT PUMPS

FIG. 2.—PUMPING CHARACTERISTICS.

the water level in the mixing tank should rise above a predetermined level.

The raw water pumps can be operated singly or in combination. Fig. 2 shows diagrammatically the head characteristics of the several combinations with respect to the probable head conditions that will vary as the reservoir level rises and falls. The operator will select one or more pumps that have combined free capacity slightly greater than the desired filtering rate, and the discharge will then be automatically throttled by the water-level control-device closely to follow the filtering rate. It will be seen that a single pump, or combination of two units, will give any capacities likely to be required. One unit will be held in reserve as a spare.

The casings of all three pumps are identical, two units having impellers of the same capacity, the third having a larger impeller. All three motors are 5-horsepower, which is sufficient to permit a future increase in capacity of the smaller units, merely by enlarging the impellers. Across-the-line starters are conveniently mounted on a nearby wall.

Aeration and Chemical Mixing. The raw-water pumps discharge into the aeration tank where vigorous agitation and aeration may be accomplished by means of diffused air introduced through porous tubes. Raw water enters near the bottom of the tank at one end and flows out into the mixing tank near the top at the outer end. Its passage through the tank is very turbulent due to the rising air bubbles with which the churning water is in contact during the detention period. Air is supplied by a Pottstown rotary positive-pressure blower, motor-driven through a U. S. Varidrive unit, by means of which the quantity of air can be regulated within wide limits by speed variations.

The diffuser tubes, 12 in number, have an internal diameter of 3 in. and a length of 24 in.; the permeability rating lies in the range between 30 and 45 c.f.m. dry air at 70° F. per sq. ft. of tube area at pressure equivalent to 2 in. of water. Tubes are mounted horizontally in sets of four at the lower end of a vertical air pipe connected with a horizontal air header above the water surface. The depth of submergence of the tubes and the resulting air pressure can be varied by changing the length of the vertical pipe in case it proves desirable to find by experiment the most effective depth. The capacity of the blower varies not only with speed but also with the pressure against

which it operates, due to the slip that is inherent in this type of blower. The blower capacity at various speeds and pressures may be found by reference to the curves on Fig. 3.

The maximum capacity of the blower at 7-lb. pressure (tubes near tank bottom) is about 70 c.f.m. of free air. This is equivalent

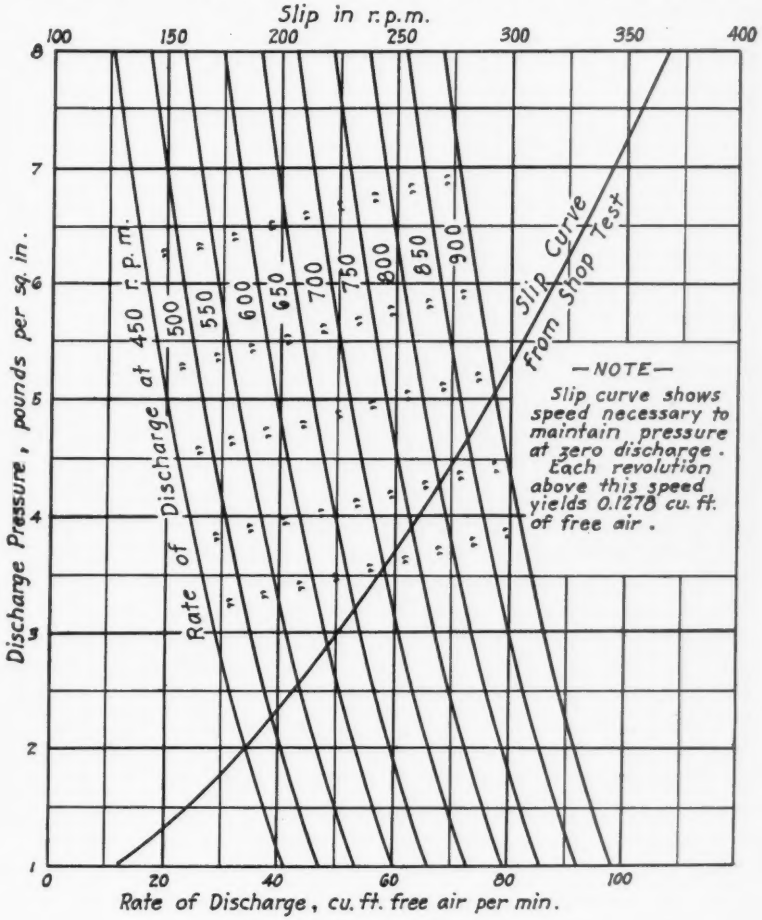


FIG. 3.—CAPACITY OF BLOWER.

to about 0.05 c.f.m. per gallon of water at the 2-m.g.d. rate, or 1 c.f.m. per sq. ft. of tank surface, which is believed to be more than adequate to accomplish all that can be expected of aeration at this plant. The blower can be slowed down to deliver a minimum of about 17 c.f.m. or about one-quarter of the maximum, providing great flexibility in the use of the aeration facilities.

After aeration, the water flows through the mixing tank where it is subjected to gentle agitation by the action of a large motor-driven wooden paddle, rotating at speeds that can be varied at will from 0.5 to 2 r.p.m. by manipulation of the U. S. Varidrive unit which drives the slow-moving bevel gear assembly from which the paddle is suspended.

Chemical Feeders. Activated carbon, alum, pre-lime (when required) and post-lime are fed by Type CFM-0-30 Syntron machines, supplied through extension hoppers from the chemical storage room on the floor above. From the first three machines the chemicals are flushed through movable hose lines to any desired point in either the aeration or mixing tanks. Lime from the last machine is flushed to a connection with the main filtered-water header through a float-fed water-seal tank.

The correct settings of the feed machines, as with most volumetric machines, are determined by trial and must be occasionally checked not only because of variation in different lots of chemicals as to dryness and density but also because of occasional partial stoppages by bits of paper, string, etc. Checking is done at frequent intervals by catching and weighing the material fed by each machine in a given time and comparing this with the intended dose.

Fig. 4 has been devised to provide a convenient means of comparing the weighings with the customary terms for dosage. For example, if it is desired to apply a given chemical at the rate of 1 grain per gal. (or its equivalent in p.p.m. or lbs. per m.g.), and the rate of flow through the plant is being controlled at 1 m.g.d., the diagram is entered at the corresponding points at the bottom and right margins respectively, and the point of intersection of the coördinates is determined; from this intersection a line parallel to the sloping lines is carried to the top or left margin where the scale indicates the desired dose to be about 45 grams per minute, or about 143 lbs. per day. The former is useful for comparison with small samples caught over

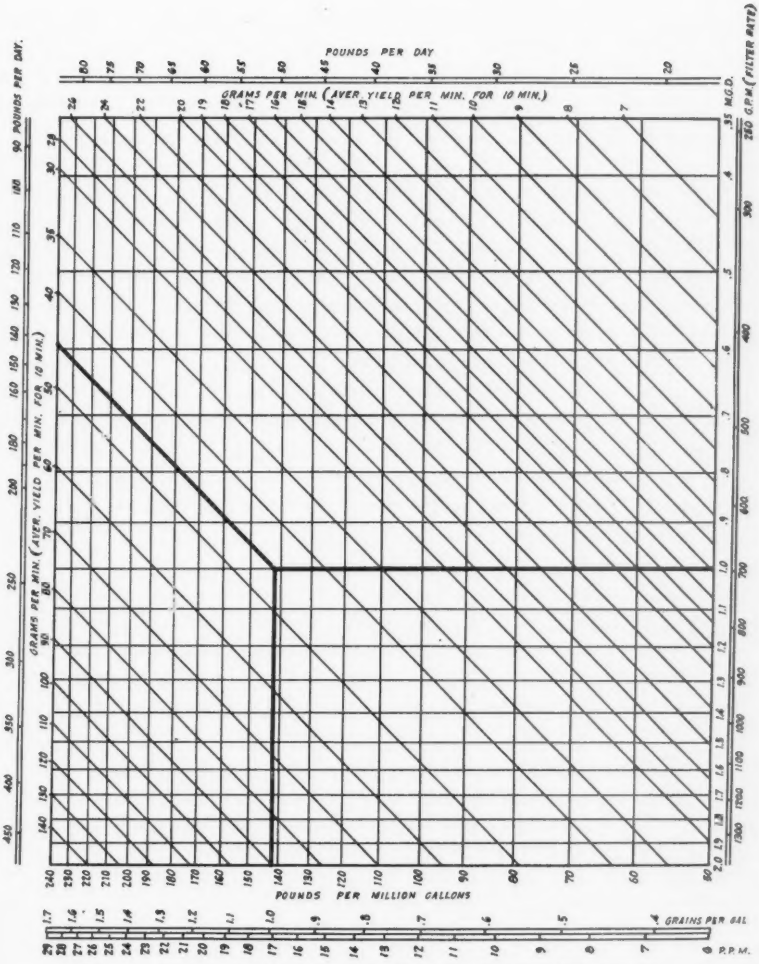

POUNDS PER DAY
GRAMS PER MIN. (AVER. YIELD PER MIN. FOR 10 MIN.)
M.G.D.
G.P.M. (FILTER RATE)
POUNDS PER DAY
GRAMS PER MIN. (AVER. YIELD PER MIN. FOR 10 MIN.)
POUNDS PER MILLION GALLONS
GRAINS PER GAL.
P.P.M.

FIG. 4.—CONVERSION DIAGRAM FOR DRY-FEED MACHINE CALIBRATION.

a period of a few minutes and the latter in checking total consumption of chemicals at the end of a week or a month.

Chlorine may be added in the mixing tank or in the suction of

the high-lift pumps, or both, by means of two Wallace & Tiernan manual-control vacuum solution-feed machines located in a special room off the chemical storage room, fully equipped with chlorine scales and manifold; there is also space for a future ammoniator if experience should indicate the desirability of this feature.

Sedimentation. From the mixing tank the treated water flows to the duplicate coagulation basins through a channel in the upper part of the headwall of the latter. The basins are of concrete, earth covered, and each has a wooden baffle lengthwise, causing the water to pass the length of the tank, around the end of the baffle, returning to the headwall on which the outlet weirs are located. Discharging into the intermediate conduit in the headwall, the settled water flows in a 16-in. pipe to the filter pipe-gallery.

For periodic cleaning, the coagulation basins will be gated off one at a time, drained through the lower conduit in the headwall and flushed out by hose taken in through access manholes provided for the purpose. Sludge and washings are discharged through the main plant drain into the tailrace below the dam and thence are carried to Great Brook and tidewater a short distance below.

Submerged lights at manholes near the outlet ends of the basins facilitate observation of the clarity of the settled water or the detection of the presence of undue amounts of unsettled floc.

Filtration. The filters are four in number, two on either side of a central gallery. Each is fully equipped with hydraulically-operated influent, effluent, drain and wash valves controlled from American Water Softener Co. operating-tables. The flow through each filter is controlled by a Builders Iron Foundry self-contained rate-controller, and recording rate-of-flow and loss-of-head gages of the same make are provided on the operating tables furnishing visual evidence of the behavior of the units in the absence of the operator.

The pipe gallery between the filters, which leads off the main pump room, is conveniently accessible for observation and adjustment of hydraulic valves, rate controllers and other equipment. The filter underdrain system consists of gravel-covered 3-in. cast-iron pipes spaced 6 in. on centers and perforated with 5/16-in. staggered holes every 3 in.

The sand, dredged from the sea near Provincetown, has an effective size of 0.43 m.m. and a uniformity coefficient of 1.4; it was laid

originally 33 in. in depth, with a view to eventually discarding about 3 in., as fines and extraneous materials, such as bits of shells and particles of seaweed, become segregated on the surface after successive washings. Adjustable spotlights provide beams that can be projected into the water on the filters for convenient observation of the water conditions.

Wash water is supplied from the filtered-water reservoir by either of two Allis-Chalmers centrifugal pumps, with capacities of 3,300 g.p.m. each, driven by 30-horsepower squirrel-cage motors. These units are stopped and started by pushbuttons, both at the switchboard and near the motors. In the wash-water header is a 16-in. Builders Iron Foundry rate-controller which throttles the wash-water pump discharge to the desired rate. The actual operating head will vary with the level at which water happens to stand in the filtered-water reservoir from which wash water is drawn, but the pump characteristics are such that a wash rate equivalent to 30 in. of vertical rise per minute or more can be supplied, if the filtered water reservoir is half full or better, and a rate of 24 in., or over, under any circumstances. Dirty wash-water is discharged through the main plant drain to the nearby tailrace leading to Great Brook and tidewater.

Filtered Water Reservoir. After filtration, the water flows to the filtered-water reservoir through a 20-in. pipe which passes beneath the pump room and to which the suctions of both high-lift and wash-water pumps are connected. The reservoir is of reinforced concrete, earth covered, and has a capacity of 540,000 gal. The roof is of flat-slab construction, and the floor which lies somewhat below ground water level is of beam-and-slab construction designed to resist upward pressure when empty.

Two access manholes are provided, and an 8-in. cast-iron float well encloses a float connected with a Builders Iron Foundry Chronoflo telemeter transmitter which actuates the water-level recorder in the filter building, informing the operators at all times as to the depth of filtered water in storage.

High-Lift Pumping. Three high-lift pumps are provided in the pump room on the ground floor of the new plant. Two are new Allis-Chalmers units driven by synchronous motors; the third is a DeLaval unit with squirrel-cage motor, comparatively new, moved from the old pumping station. Water is drawn from the main pipe connecting

the filters with the filtered-water reservoir and delivered into the distribution sytem through a Builders Iron Foundry Venturi meter, the register-indicator-recorder of which is on the instrument panel in the new building. The synchronous motors of the two new units, 50-hp. and 75-hp. respectively, may be operated with 0.8 leading power factor with corresponding advantageous effect upon electrical conditions.

A Nash Hytor vacuum pump and vacuum tank serve to keep the high-lift pumps primed at all times. The same vacuum system also serves the wash-water pumps. Although this system is normally separate from the vacuum system which primes the raw-water pumps, the Nash units are arranged and valved so that either may be connected to serve either vacuum tank in an emergency, thus avoiding the necessity of providing spare units.

The head characteristics of the high-lift pumps are shown on the lower part of Fig. 2 together with their relation to the system head with one or more pumps in use and with fluctuating water levels in the standpipe and filtered-water reservoir. Pumps will usually be used singly, being selected approximately to follow consumption and keep the standpipe substantially full, but they may be run in pairs if needed, the third unit being held in reserve. These pumps are started and stopped by nearby pushbuttons communicating with the controllers located in the switchboard on the floor above; pushbuttons for stopping only are also provided at the switchboard.

A Builders Iron Foundry Chronoflo water-level transmitter, located in a manhole at the foot of the distant standpipe, drives the telemeter on the panel in the plant, recording the standpipe water-level, so that the operators may properly regulate pumping operations.

Connection of the high-lift discharge pipe with the existing pumping station is preserved so that, by the manipulation of valves, raw water can be pumped direct from the old station to the distribution system in an emergency. The cross-connection is provided with two gate valves, both of which are normally closed, and with an open drip cock in the short section of pipe between them, thus insuring against accidental leakage of untreated water into the filtered-water lines.

Electric Power System. The Borough of Groton, through its Water and Electric Department, operates the Groton electrical system. Power is obtained at wholesale from a nearby high-tension trans-

mission line and distributed from the Department's sub-station. The filter plant is supplied from this system. The incoming power line, carrying 2300-volt, 3-phase, 60-cycle current enters the building underground from a nearby pole and terminates at an oil circuit breaker in the General Electric dead-front switchboard on the second floor. Through this breaker are energized the General Electric Pyranol power and light transformers located in a special room on the ground floor. There are three 75-kva. power transformers and one 10-kva. lighting transformer, all for 2400 volts to 120/240 volts with alternative taps for adjustment.

From the power transformers, 230-volt circuits are carried (1) to a power panel on the first floor from which power is distributed to the small motors of the raw water and vacuum pumps, the air compressor, the mixing tank and the chemical hoist, and (2) back to an oil circuit breaker in the main switchboard which energizes the low-voltage bus in the switchboard superstructure. This bus feeds the remaining sections of the switchboard consisting of controls for the squirrel-cage motors of the two wash-water pumps and the smallest high-lift pump, and for the two synchronous motors of the larger high-lift pumps. The end section of the switchboard houses a truck and a spare oil circuit-breaker suitable for replacing either the incoming line unit or the transformer secondary unit.

The small transformer supplies 115-volt, single-phase current not only for the lighting system, but also for the oil burner, unit heaters, chemical feeders, recording instruments, etc.

Emergency Power. With the inauguration of the new treatment plant it was felt that an auxiliary source other than the local system was desirable even though the high-tension line feeding the Groton sub-station is in duplicate. Therefore, a used General Electric 75 kva., 2300-volt, 3-phase, 60 cycle generator with 6.6 kw. direct-current exciter was purchased and installed in the Diesel pump room of the old pumping station. The generator and exciter are separately belt-driven from the existing line shaft which is directly connected to the 55-hp. water turbine. This line shaft may also be powered through a belt-driven line shaft clutch by an existing Diesel engine rated at 115 hp., 260 r.p.m. The generator and exciter are connected, through a two-panel switchboard located in the same room, to the existing old pumping-station switchboard bus through a set of fused disconnect

switches. Thus energy from the auxiliary generator can be supplied to the treatment plant primary feeder which can be isolated from the normal supply line by opening an oil switch mounted on one of the poles of the incoming line.

By using the water wheel, sufficient electricity can be generated to supply the filter plant during the low consumption (winter) months; and by using the Diesel engine, sufficient power can be furnished to run the plant at full capacity whenever necessary. Of course, the use of the water turbine cannot be long sustained except in those seasons when the flow of Great Brook sufficiently exceeds the requirements for water supply.

The usefulness of this auxiliary source of power was well demonstrated during the severe sleet storm of March 4, 1940, when outside power was lost for about 12½ hours. The generator installation had not been fully completed, but was temporarily connected within a few hours during which time the standpipe carried the load. The generator was then put into service, driven by the water wheel, per-

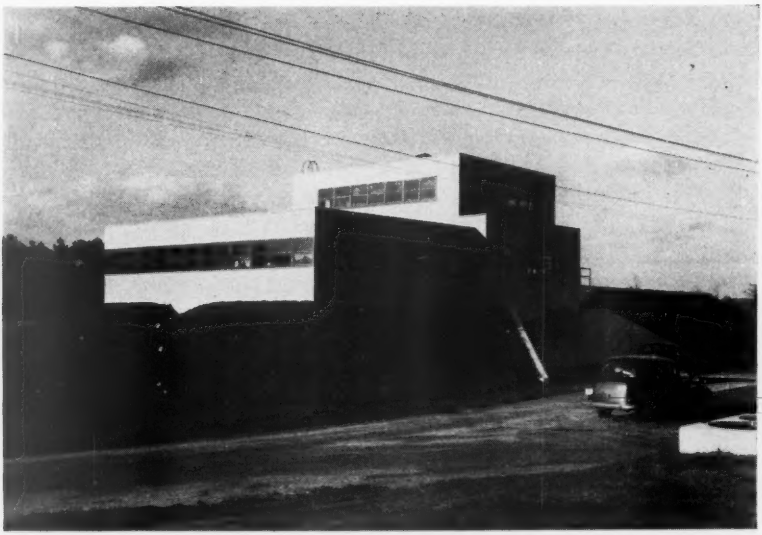


FIG. 5.—EXTERIOR OF BUILDING, GROTON WATER PURIFICATION WORKS.

mitting the operation of the filter plant without further interruption or curtailment of output, until outside power was restored.

Building Superstructure. The new plant is housed in a single structure which is fireproof throughout. The exterior, of smooth cream brick, is severely plain, but of interesting lines. (Fig. 5) The windows have steel sash and are liberally provided with ventilating sections; window areas are large, particularly in the filter wing, three sides of which have continuous windows, thus providing excellent light for observation of filter operations. The interior of walls, and partitions, is of buff "utility" tile, 5 in. by 12 in., which is inexpensive but attractive and easily cleaned.

On the ground floor, the main room is occupied by the pumps for raw water, wash water, and high service. In this room are also located the automatic priming-systems and the oil-burning steam-heating boiler; and from it lead off the filter pipe-gallery and the transformer room. Beneath the pump room is an unfinished basement for access to the various pipe lines that pass through this area.

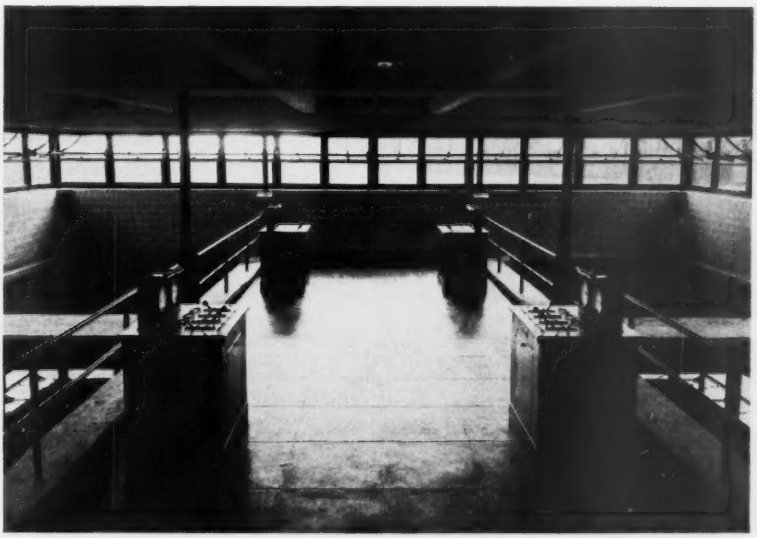


FIG. 6.—FILTER OPERATING GALLERY.

On the second floor are located the main switchboard room, the filter-operating wing (Fig. 6), the office and laboratory, the toilet, the mixing-tank room and an entry for delivery of chemicals. A driveway is sloped up to the rear of the building, permitting a truck to be backed up to a large doorway, the sill of which is approximately at truck-bed level. A truck may be quickly unloaded into the entry, where the chemicals will be under cover from the weather and can then be hoisted at any convenient rate or time into the storage room above through a trap door by means of an electric hoist.

The third floor contains the chemical-storage room and the chlorine room. In the former a monorail permits the electric hoist, by means of which chemicals are brought up from the unloading entry, to be moved with its load to convenient points for storage. From this floor chemicals are charged directly into the extension hoppers leading to the dry-feed machines in the mixing-tank room below. The chlorinators are in a separate room with high door sill and ventilator opening out-of-doors near the floor line, so that an accidental spill will be largely prevented from reaching other parts of the building.

Heating is by steam, unit heaters being used in all spaces except the main entrance, toilet, and chlorine room, where cast-iron radiation is employed.

Construction Cost. The construction work was done under two contracts on both of which N. Benvenuti & Sons of New London, Conn., was the low bidder. In order to meet P.W.A. requirements for starting construction on very short notice, the filtered-water reservoir was selected from the initial contract. This unit is located somewhat apart from the rest of the work and so could be conveniently handled as a separate contract; furthermore, it could be designed rapidly due to its relative simplicity.

The work of design was started July 6, 1938 and Contract 1 was submitted to P.W.A. July 20 and advertised August 1; construction was started August 23 and completed November 23. Contract 2 included all remaining work and was submitted to P.W.A. October 27, 1938 and advertised October 31; construction was started November 26, 1938 and completed September 23, 1939.

The contract costs were as follows:

Contract 1.	Filtered water reservoir	$ 13,230.40
Contract 2.	Filter and pumping plant, etc.	126,786.50
Total		$140,016.90

Staff. One operator is on duty in each of three shifts; a fourth acts as relief man and as spare hand when not on relief duty. The day operator is trained to conduct routine laboratory work, and the whole plant is, of course, also under the eye of the waterworks superintendent. The size of this staff is not essentially different from that formerly required for pumping alone before the filter plant was constructed, so that the installation of the new plant has not materially affected labor costs.

Use of Chemicals. The rates of application of chemicals have naturally varied with seasonal and other changes in the raw water. Monthly averages for the first year of operation are given in Table 1.

TABLE 1—SUMMARY OF CHEMICALS USED

Month	Alum	Lime		Carbon	Chlorine	
		pre	post		pre	post
		Pounds per m.g.				
1939						
Oct.	77	0	46	21	7.5	2.4
Nov.	89	0	42	9	7.9	1.8
Dec.	186	0	93	0	5.3	3.0
1940						
Jan.	200	0	124	0	0	3.4
Feb.	159	33	86	0	0	3.2
Mar.	161	42	49	0	0	3.1
Apr.	148	33	39	0	0	3.5
May	164	23	47	0	0	3.5
Jun.	208	33	77	0	10.1	2.8
Jul.	191	22	91	0	9.7	3.1
Aug.	174	0	101	11	11.2	3.2
Sep.	156	0	91	10	10.8	3.2

Filter Operation. Water consumption has been such that the filter plant has thus far been operated at less than full capacity; individual filters are usually operated two or three at a time but are used in rotation. When a unit requires washing, a fresh unit is cut in, the dirty unit is washed and then held in reserve until its turn again comes. Table 2 shows a summary of filter operations for the first year.

Corrosion Control. The use of alum, as would be expected, results in an increase in carbon dioxide in the treated water. Lime is therefore continuously fed to the filtered water to reduce the aggressive

TABLE 2—SUMMARY OF FILTER OPERATION

Month	Total filtered, m.g.d.	Per cent wash water	Net yield, m.g.d.	Rate of filtration, m.g.a.d.	Av. length of filter runs, hr.	Av. loss of head, ft. initial	Av. loss of head, ft. final
1939							
Oct.	0.708	4.6	0.676	99	24.9	0.95	10.9
Nov.	0.786	4.8	0.748	102	24.2	1.15	10.6
Dec.	0.762	3.9	0.733	107	30.3	1.25	10.0
1940							
Jan.	0.806	3.7	0.776	117	32.3	1.3	9.5
Feb.	0.713	3.2	0.690	110	38.0	1.26	9.5
Mar.	0.759	3.0	0.736	116	39.3	1.22	9.7
Apr.	0.764	3.4	0.738	106	41.3	0.94	9.9
May	0.786	4.4	0.751	124	26.8	0.96	9.6
Jun.	0.848	4.8	0.808	124	25.9	0.91	9.9
Jul.	0.961	4.7	0.916	124	25.0	0.97	9.6
Aug.	1.132	5.2	1.073	115	25.0	0.73	10.3
Sep.	1.132	4.4	1.082	124	20.5	0.99	10.0

character of the filter effluent. Because of the large use of water for locomotive boilers by the New Haven R.R. at its freight, coaling, and watering point in Groton and the consequent importance of avoiding undue hardness, it has seemed prudent to use somewhat less lime than might otherwise be desirable. The addition of lime has therefore not only been regulated to raise the pH to a value that will control corrosiveness but has also been limited to keep the resulting hardness below 50 p.p.m.

It was anticipated that the sudden change in water characteristics, when filtered water was first pumped into the pipes of the distribution system, might result in temporary trouble and complaints due to the dislodgment of pipe slimes, etc. that had accumulated in the years of contact with untreated water. However, there has been practically no difficulty from this source, although a few isolated minor complaints have come in. It has also been observed that old tubercles on the interior of pipes have noticeably softened, and it is to be expected that this may eventually lead to some sloughing, requiring occasional flushing of dead ends until stability is reached. Thereafter it is anticipated that "red water" or similar troubles will be at an end.

Taste and Odor Control. Aeration and activated carbon were

both used at the outset with successful results in the control of tastes and odors that were prevalent in the raw water in the early fall of 1939. Both were later discontinued, as the water temperature dropped and algae growth diminished, and were not resumed until the late summer of 1940. There has been no evidence of taste or odor from winter organisms; the operators are vigilant in their watch for these, and aeration or carbon, or both, can be quickly resumed if necessary. Thus far, taste and odor in the plant effluent has been adequately controlled; a very few instances have been reported in the distribution system, but these are believed to be of temporary character related to the transition from untreated to filtered water.

Color Removal. Raw water color usually runs about 30 or 40 p.p.m., although it has sometimes been much higher when the reservoir is influenced by quick runoff from some of the marshy areas on the watershed. It has been possible consistently to reduce the color of the filter effluent to about 5 p.p.m. which is negligible so far as appearance is concerned.

Chlorination. At the outset, chlorine was applied both to the raw water and the filtered water as a precaution during the period of tuning-up the plant and training operators. Pre-chlorination was omitted during the winter and spring but was resumed when it was found to be advantageous in assisting treatment reactions and control of organisms in the summer of 1940. Post-chlorination at the suction header of the high-lift pumps is regulated to maintain a chlorine residual of at least 0.1 p.p.m.

Laboratory Work. The room on the second floor assigned as an office and laboratory is furnished with laboratory benches and equipped with apparatus for the simpler physical and chemical determinations that are necessary for adequate routine control of the treatment process. For the present, it is not planned to attempt to do biological work at this laboratory, other than simple microscopic observations. Daily tests are made and tabulated; a summary of the average results is shown in Table 3.

Cost of Operation. Additional operating costs over those prior to filtration consist of the cost of chemicals and of power (other than for high-lift pumping). Labor, as already stated, is essentially unchanged, and fuel for heating should not be much different from that previously required in the old pumping station. Additional power

TABLE 3—SUMMARY OF PHYSICAL AND CHEMICAL DETERMINATIONS

Month	Color p.p.m. Raw	Color p.p.m. Tap	pH Value Raw	pH Value Tap	CO_2 p.p.m. Raw	CO_2 p.p.m. Tap	Alkalinity p.p.m. Raw	Alkalinity p.p.m. Tap	Hardness tap p.p.m.	Free chlorine p.p.m.
1939										
Oct.	30	6	6.7	7.1	3.6	1.8	9.2	11.5	32.6	0.12
Nov.	31	7	6.7	7.1	3.5	2.3	8.3	10.8	28.6	0.11
Dec.	39	11	6.7	7.3	4.5	2.4	6.5	11.2	36.4	0.11
1940										
Jan.	40	6	6.3	7.3	10.3	3.3	7.3	17.7	40.8	0.13
Feb.	41	6	6.1	7.1	12.1	3.3	6.7	16.6	39.1	0.12
Mar.	37	5*	6.3	7.2	9.0	2.5	5.0	10.8	32.3	0.12
Apr.	41	5*	6.6	7.3	6.3	1.6	4.8	8.3	29.1	0.11
May	55	5*	6.7	7.2	6.5	2.0	6.2	9.1	34.9	0.12
Jun.	80	5*	6.5	7.3	10.7	2.2	7.2	13.5	43.7	0.12
Jul.	72	5*	6.5	7.4	13.1	2.3	8.5	16.0	45.6	0.12
Aug.	67	5*	6.6	7.4	9.1	2.2	9.6	16.7	45.5	0.12
Sep.	42	5*	6.8	7.6	5.0	1.4	9.2	15.2	42.9	0.13

*Less than 5 p.p.m.

requirements amount to about $1.75 per m.g. of net yield. Chemical costs are shown in Table 4.

TABLE 4—COST OF CHEMICALS, PER MILLION GALLONS OF NET YIELD

Month	Alum	Lime	Carbon	Chlorine	Total
1939					
Oct.	$1.19	$0.29	$1.37	$0.80	$3.65
Nov.	1.38	0.27	0.44	0.76	2.85
Dec.	2.84	0.60	0	0.64	4.08
1940					
Jan.	3.06	0.79	0	0.26	4.11
Feb.	2.35	0.77	0	0.24	3.36
Mar.	2.45	0.58	0	0.24	3.27
Apr.	2.26	0.46	0	0.26	2.98
May	2.53	0.45	0	0.33	3.31
Jun.	3.19	0.74	0	1.01	4.94
Jul.	2.94	0.73	0	0.95	4.62
Aug.	2.68	0.68	0.80	1.13	5.29
Sep.	2.39	0.61	0.74	1.09	4.83
Average	2.44	0.58	0.28	0.64	3.94

Prior to the construction of the filter plant, the water was heavily chlorinated at the pumping station, at rates averaging about 15 lbs. per m.g. equivalent to a cost of about $1.12 per m.g. If this amount is subtracted from the average total cost of chemicals, the net additional cost will average about $2.82 per m.g.

The availability of an ample supply of clear, sparkling, palatable water cannot fail to prove advantageous to the community and, in the long run, to justify the expense involved.

Administration. The construction project was administered for the Borough of Groton by a joint committee composed of the Warden and Burgesses of the Borough and the Water and Electric Commissioners; a bronze tablet at the head of the entrance staircase gives recognition to these officials. In active executive charge, representing this committee, was the Superintendent of the Water and Electric Department, who is a co-author of this paper. Mr. Milton A. Saunders was Resident Engineer representing Metcalf & Eddy, Engineers, who designed the new works. The operation of the plant has been added to the other duties of the Superintendent, Metcalf & Eddy having had advisory supervision during the first six months of operation, assisting in the initial adjustment and tuning of the plant, in the instruction of operators and in the systematizing of methods and plant records.

OPERATION OF THE CAMBRIDGE, MASS., FILTRATION PLANT

BY FRED E. SMITH*

[*Read November 21, 1940.*]

The Cambridge Water Purification Plant was placed in service in 1923. A description of the original plant has been previously presented before this Association (1). Alum was used for coagulation. This treatment was accompanied by sedimentation, and followed by rapid-sand filtration, aeration, and disinfection. Two papers (2, 3) recording operating experience during a period of 8 years, have been published. This article will record details of the plant enlargement program completed in 1932. In addition, operating experiences since 1930, which may be of interest to others engaged in water purification, will be related.

Plant Enlargement Program. The original plant contained 10 filters, each having a capacity of 1.4 m.g.d. In 1929 and 1930, daily consumption of water in Cambridge had so increased that it was occasionally necessary to supplement the filtered supply with water from Fresh Pond, treated by chlorination. In 1931, construction of 6 additional filters was started. These new units were placed in service in 1932. They duplicated construction features of the original filters, being 20 ft. by 24 ft. in area and having a capacity of 1.4 m.g.d. The Wheeler bottom, which had rendered satisfactory service in this plant, was again used. Provision for future enlargement had been incorporated in the original building, and foundations for the new units were in place. Also, connections that had been provided in the wash water and drain pipes enabled the installation of the new piping to be readily made. The filter building was enlarged to cover the new units, which were built on both sides of a pipe gallery constructed at right angles to the original gallery.

Cape May sand and gravel were used in the new filters. The depth of sand was 27 in., supported by 9 in. of graded gravel. The effective size of the sand was 0.47 m.m., and the uniformity coefficient

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was 1.26. The additional filters increased the plant capacity to 22.4 m.g.d., a volume that will be ample for this city for many years. The cost of the new construction was $98,081.

New Pumps. During the drought of 1930 and 1931, the volume of water in the Cambridge reservoirs had been greatly depleted. In order to add to the available supply, two centrifugal pumps, each of 7.5-m.g.d capacity, were installed in a new building on the shore of Fresh Pond. Two intake pipes were laid, one at a depth of 32 ft. and a second at a depth of 22 ft. This arrangement allows 800 m.g. of water, previously unavailable, to be pumped to the filtration plant. The cost of the pumps, intakes and building was $48,574.

Water Consumption. That enlargement of purification facilities in Cambridge was necessary is indicated by examination of filtered-water records. In each year of operation since 1934, there have been one or two months in which the average volume of filtered water has exceeded 14 m.g.d., the capacity of the original plant. In 1939, there were 5 months in which more than 14 m.g.d were filtered; in August of that year the average production was 15.25 m.g.d. The volumes filtered on 3 successive days in August, 1939, were 17.49, 17.94, and 17.77 m.g. October, 1940, contained 27 days during which the volume of filtered water exceeded 14 m.g. Thus, an enlargement of filtering capacity was imperative.

New Operating Schedule. Before the plant was enlarged, the original filters were in operation 24 hours daily, being removed from service only for washing, and placed in service immediately after completion of the wash. This operating program was necessary in order to provide sufficient water to meet the demands of the city. When the new filters became available for duty, the operating schedule at the pumping station was changed to a single period of 15 to 16 hours at the rate of 20 m.g.d. This new pumping program necessitated the operation of 12 filters for approximately 15 hours daily, and 4 or 5 filters during the balance of the 24 hours. The normal rate of plant operation, therefore, is 16.8 m.g.d., and on days of highest consumption this figure is increased to 18.2 m.g.d.

Reservoirs. Water flows by gravity from Stony Brook Reservoir to the filtration plant and Fresh Pond, from which water is pumped to supplement the gravity supply for approximately 13 hours daily. Pumpage is necessary because the volume of water from Stony Brook

Reservoir is restricted to a steady flow at the rate of 14 m.g.d. As the filtration plant normally requires water at the rate of 16.8 m.g.d., the pumped supply is necessary. Raw water delivered to the filtration plant, therefore, consists of a mixture of 75% Stony Brook water and 25% from Fresh Pond. During night operation the gravity supply is of sufficient volume for the number of filters in service.

Quality of Raw Water. Stony Brook Reservoir, being relatively small and having a comparatively large catchment area, delivers water of high color and large numbers of bacteria whenever there is heavy runoff. The addition of Fresh Pond water, which has been subjected to a long period of storage, reduces peaks of color and higher bacterial numbers of the gravity supply. The highest color of the mixed water during the past 5 years was 65 p.p.m., the minimum 17 p.p.m. Turbidity is low, being 2 or 3 p.p.m. A musty odor is usually present in the raw water. The bacterial content occasionally exceeds 1000 on gelatine accompanied by several hundred on agar. The coliform test has yielded positive results occasionally in 1 c.c. of sample, and rarely in 0.1 c.c. of sample. However, there are many days on which the test for coliform organisms is negative in 10 c.c. of raw water. The bacterial load upon the filtration plant is normally not heavy. Table 1 lists

TABLE 1—RAW WATER ANALYSES.

Year	Color p.p.m.	Alkalinity p.p.m.	pH	Bacteria per cc. gelatine	agar	Indicated coliforms per 100 cc.
1935	27	19.0	7.0	190	61	24.2
1936	27	16.5	7.0	255	71	33.1
1937	33	18.5	6.8	123	47	62.2
1938	41	19.5	6.8	104	47	61.0
1939	29	19.5	6.9	179	43	22.0

analytical results on raw-water samples, yearly averages being recorded.

The results of color determinations and bacterial numbers have been lowered by periods of pumpage when the entire plant supply was taken from Fresh Pond. Its water contains less color and fewer bacteria than does the gravity supply.

Copper Sulfate Treatment. From 1931 until 1936 the reservoir treatment for microscopic organisms was not required. In August, 1936,

Synura was present in Stony Brook Reservoir. The organisms were concentrated 10 ft. below the water surface. Their number was small, being 12 standard units per c.c. The reservoir was removed from service, and copper sulfate was applied in a dosage of 3 lb. per m.g. The chemical was suspended in bags from a boat powered by an outboard motor. The treatment successfully killed the organisms, and the reservoir was returned to service after 6 days. This experience with *Synura* differed from an earlier one in 1931, because the intensity of odor and taste was considerably less in 1936. Cambridge water quickly acquires objectionable odor and taste from a few units of *Synura*. Immediate treatment is applied when the first indications of odor or taste are observed, in order to prevent delivery of unpalatable water to consumers' taps.

In November, 1936, a growth of *Melosira* occurred in Stony Brook Reservoir. There were present 673 standard units of diatoms per c.c. No odor was observed, but filter service had been decreased to 11 hours. Copper sulfate dosage of 2.2 lb. per m.g. killed most of the organisms, and after 3 days, periods of filter service had increased to 14.9 hours. The reservoir was not removed from service during treatment. Another growth of *Synura* in Stony Brook Reservoir necessitated treatment in 1938, 2 lb. of copper sulfate per m.g. being employed as the effective dosage.

Coagulation. The operation of the plant for several hours daily at a rate of 16.8 m.g.d. reduces the period of detention in the sedimentation basin to 2.14 hours. The efficiency of coagulation and sedimentation is lessened, and the purification load on the filters is consequently increased. The improved coagulation that would be provided by additional sedimentation capacity is indicated by the results of operation during night hours. Stony Brook water, which always presents greater difficulties in coagulation than does Fresh Pond water, is coagulated satisfactorily when 4 or 5 filters are in service at night. Based on 5 filters in operation the period of detention in the basin at such times is 5.14 hours. The addition of Fresh Pond water to the gravity supply results in improved coagulation. The greater alkalinity of the pumped supply supplements the comparatively low akalinity of Stony Brook water, so that the addition of an alkali to the raw water has not been necessary.

The sedimentation basin is divided by a center wall into two bays,

one of which may be removed from service for cleaning while the other bay is in operation. The basins are cleaned at intervals of 3 weeks in summer, the period of service being extended to 6 weeks in cold weather. The shortened period of service in summer is caused by increased efficiency of coagulation and sedimentation and the greater volume of water being treated. The upper layers of water are drained by gravity into Fresh Pond, and the sludge is pumped to the city sewer.

Designs for the erection of a new sedimentation basin that, with the present unit, will provide for a detention period of 6 hours have been prepared. Facilities for mixing of coagulant and raw water have been incorporated in the plans. At present, the dosed water is mixed to a limited extent during passage through the 42-in. raw water main to the inlet chamber of the sedimentation basin.

Alum Dosages. Table 2 records the amounts of alum used in

TABLE 2—ALUM DOSAGES

Year	Alum lb. per m.g.
1935	159
1936	156
1937	185
1938	193
1939	159

treatment of Cambridge water, average figures being listed for the past 5 years.

Considerable rain fell in 1938, and the higher color of the raw water is reflected in the average amount of alum applied in that year. Color removal by coagulation averages 15% on a yearly basis. In summer the removal is increased to 50%, but this figure is greatly reduced during winter months.

Alum Equipment. As a part of the plant enlargement program, the solution-feed apparatus used for the application of alum to the raw water was superseded by dry-feed equipment. Two Hamblet dry feeders were installed. Features of the Hamblet apparatus are its rugged build and simplicity of operation. The machines are powered hydraulically, using city pressure water available in the building. A storage hopper of 2450-lb. capacity was installed over each machine, and an electric hoist is used to raise the bags for dumping. The broad

range of alum feed provided by the machines has proved to be adequate for raw-water requirements. The lump alum formerly used with the solution-feed apparatus was discontinued, and a rice size was selected. The new size of alum dissolves readily irrespective of water temperature or rate of dosage. The 2½-in. lead solution-line formerly in use was abandoned, and the new machines were connected to a 4-in. Universal cast-iron pipe. This line delivers alum solution to the point of application in the raw-water main, a distance of some 300 ft. Accumulation of inert material occurs in the pipe to a moderate extent. Remedial measures employed are frequent flushing and the use of hand-cleaning methods at intervals of several years. The performance of this dry-feed equipment has been very satisfactory for more than 8 years.

Filters. After the 6 new filters had been placed in operation, the 10 original units were overhauled. The cement balls of the Wheeler bottom had spalled off and deteriorated. These were replaced with baked clay balls similar to those used in the new filters. Clogged nipples were re-opened and the lateral underdrains were inspected. A few laterals required cleaning. When the sand was replaced in these 10 filters, the required quantity was added to restore the surface to a point 24 in. below the top of the wash-water troughs. This procedure necessitated the purchase of additional Ipswich River sand which was added to 2 filters. The new sand was relatively fine, the effective size being 0.35 mm. At present, the 2 filters contain sand of 0.43-mm. effective size, representing a combination of old and new sands. All filters have rendered efficient service since 1932, notwithstanding the additional purification load caused by the higher rate of plant operation and the shortened detention period in the sedimentation basin. Yearly averages of filter runs have been 13.6 to 16.1 hours during the past 5 years. In that period the highest monthly average was 20.5 hours in August, 1938.

Treatment of Sand. The sand grains have increased in size due to deposits of organic matter and aluminum compounds. Table 3 lists the average results of sieve analyses of Cambridge filter sand.

Caustic treatment of filter sand is conducted yearly. Flake caustic soda is spread over the sand surface, 600 lb. being used. The chemical is dissolved by filling the filter with wash water to a level 8 in. above the sand. The solution is filtered to waste until the level

TABLE 3—AVERAGE SAND SIZES

Year	1933		1939	
	Effective Size m.m.	Uniformity Coefficient	Effective Size m.m.	Uniformity Coefficient
Old filters	0.48	1.54	0.50	1.52
New filters	0.47	1.31	0.50	1.26

is 2 in. above the sand. After a 48-hour period has elapsed, the filter is washed for 20 to 30 minutes, or until all traces of caustic have been removed. This treatment has helped to maintain the sand in a clean condition.

Filter Breaking. During the winter months, the finely divided floc produced in the coagulation of Stony Brook water penetrates into the sand and is discharged with the filtered water when the loss of head reaches 6 or 6.5 ft. At such times, the effluent contains 0.5 to 1 p.p.m. of turbidity and 10 to 15 p.p.m. of color, and occasionally the bacteriological quality is unsatisfactory. As previously stated, mixing Fresh Pond water with the gravity supply aids in the formation of a stronger and coarser floc. Another remedial measure consists of washing the filters at a lower final loss of head. Also, the expedient of removing from service 2 or 3 filters which contain coarse sand has been resorted to. In 1939, the caustic treatment of sand was delayed until November, and filter operation during the past winter was conducted with considerably less trouble due to breaking. The procedure of late caustic treatment was repeated this year in the attempt to overcome filter breaking.

Filter Washing. Wash rates in this plant are 22 to 28 in. of vertical rise per minute, depending upon water temperature. The washing period is 3½ to 4 minutes, timed after the water first overflows into the troughs. The duration of the wash period is comparatively long, and this condition is caused by the purification load borne by the filters. Yearly averages of the daily number of washings are 14 to 17, based upon results of operation since 1934. In this period the percentage of total water filtered used for washing purposes has been 3.7% to 4.6%.

Disposal of Used Wash Water. The original plant was equipped with a small receiving basin into which used wash water was delivered by gravity. Pumps returned the wash water to the sedimentation basin.

This practice was adopted as a matter of water economy, and it was also expected that pre-coagulated particles would serve as nuclei for rapid floc formation. Favorable results were not obtained, however, and the used wash water was discharged to the sewer for several years.

It became necessary to conserve water during the drought of 1930, and the practice of recirculating wash water was resumed. In 1932, a pipe line was constructed to deliver the wash water by pumpage to Fresh Pond, and this method of disposal has been in operation for more than 8 years. Water representing an average volume of 203 m.g. yearly has been conserved, and no deleterious effects upon the quality of Fresh Pond water have been observed. It was anticipated that Fresh Pond might be seeded with microscopic growths removed from the filters, but the reservoir has been remarkably free from large growths of organisms for several years.

Aeration. The riffle-plate aerator releases odors and tastes from the filtered water to a certain extent. A small amount of carbon dioxide is also removed, about 3 p.p.m. out of 10 p.p.m.

Corrective Treatment. A paper (4) has been published describing the methods that have been used at Cambridge to deliver water of low corrosive powers. Lime has been used in corrective treatment of filtered water for 10 years. The quantities of lime applied to the water have averaged 78 to 91 lb. per m.g. during the past 5 years. The effluent pH has been maintained close to 8.8, and the water has contained 5 to 10 p.p.m. of carbonate alkalinity. This treatment appears to have been successful in combating the corrosion problem in this city. Few complaints of red water have been received during recent years. These have been caused principally by reversal of flow in mains or greatly increased flow due to heavy draft at hydrants.

Lime has increased the hardness of the water to 40 to 45 p.p.m. This represents an increase of 15 p.p.m. of hardness. Consumers have not complained about this increase, and industrial users of water have offered no objections.

Chlorination. The plant continues to operate the Wallace and Tiernan chlorinator installed in 1923. The machine has performed dependably, and repairs have been minor ones. A reserve unit of earlier design is used as a spare machine. For several days, this unit was operated successfully when the regular chlorinator was being repaired. Chlorine dosages average 1.8 to 2.0 lb. per m.g.

TABLE 4—EFFLUENT ANALYSES

Year	Color p.p.m.	Alkalinity p.p.m.	pH	CO_2 p.p.m.	Turbidity p.p.m.	Bacteria per c.c. gelatine	agar	Indicated coliforms per 100 c.c.
1935	3	22.5	8.7	0	0	4	2	.01
1936	3	19.5	8.7	0	0	3	1	.07
1937	2	22.5	8.8	0	0.3	1	1	.01
1938	2	22.5	8.8	0	0.2	1	1	.13
1939	2	22.0	8.8	0	0.2	1	1	.04

Quality of Plant Effluent. Table 4 presents data illustrating the quality of plant effluent. Average analyses for 5 years are recorded.

The average color has been 2 p.p.m. for the past 3 years. Inspection of the daily records of the 5-year period indicated that the highest effluent color was 17 p.p.m., in October, 1938. The average color for that month was 6 p.p.m. The raw-water color was 65 p.p.m., an analysis of a mixture of Stony Brook and Fresh Pond water. The alkalinity results reflect the addition of lime to the filtered water; corrective treatment also influences the results of pH and carbon dioxide. The average hardness was 45 p.p.m. A faint odor was present occasionally in the plant effluent. A musty or vegetable odor is difficult to remove, and can be detected by a trained observer. No complaints of odor or taste have been received from consumers for several years. The bacteriological quality of the effluent has been excellent. Very low plate counts have been recorded, and the number of positive lactose broth tubes has been small.

Cost of Operation. In Table 5 are listed cost figures in summarized form for the years 1935 to 1939 inclusive. Also, the volumes of filtered water are recorded. The yearly cost of operation, exclusive of fixed charges, has totaled $46,000 to $47,000. The highest cost per m.g. was $10.10 in 1938. The cost of chemicals per m.g. was $3.16 in 1937, and a minimum of $2.46 in 1939. The volume of filtered water recorded for 1936 was 5045 m.g., the maximum production in 17 years of plant operation.

Laboratory. Laboratory supervision of the purification processes was established when the plant was placed in service. Facilities for control tests and special investigations were installed, and during the years of plant operation newly developed equipment and supplies have

TABLE 5—SUMMARY AND COST OF OPERATION

Year	Cost of Operation 1935	1936	1937	1938	1939	Cost per Million Gallons 1935	1936	1937	1938	1939
Chemicals	$13,803.00	$14,166.00	$15,393.00	$14,532.00	$12,279.00	$2.85	$2.81	$3.16	$3.08	$2.46
Labor, Operators	17,948.08	18,193.87	18,120.72	19,317.21	21,411.32	3.70	3.60	3.72	4.09	4.30
Laboratory, Chemists, Supplies	4,113.78	4,124.43	4,104.45	4,364.66	4,366.46	0.85	0.82	0.84	0.93	0.88
Power Fuel	10,223.79	8,445.55	7,880.87	6,573.19	8,358.75	2.12	1.67	1.62	1.39	1.67
Equipment, Service	1,572.13	1,860.55	1,286.55	2,858.82	1,293.95	0.33	0.37	0.26	0.61	0.26
Totals	$47,660.78	$46,790.40	$46,785.59	$47,645.88	$47,709.48	$9.85	$9.27	$9.60	$10.10	$9.57
m.g. Filtered	4841	5045	4865	4722	4988					

been provided. This policy has enabled the laboratory to maintain the scientific control required for the production of water meeting modern standards of quality. Research programs have included studies of new coagulants on a laboratory scale. Chlorinated copperas and ferric sulfate have been investigated. These coagulants do not yield results with Cambridge water that would justify replacement of alum on the basis of economy or quality of effluent water. The improvement of coagulation by the addition of sodium silicate was not successfully demonstrated. An investigation (5) of the comparative efficiencies of lactose broth and brilliant green bile in the bacteriological examination of Cambridge water has been published.

Personnel. Successful plant operation has been materially aided by the faithful service rendered by the filter operators and other employees. All are aware of the necessity for the production of safe water irrespective of difficulties encountered. The operators have had at least 12 years of experience in this plant.

Visitors. Plant operation was demonstrated in 1939 to 25 classes or organized groups of visitors from hospitals and educational institutions. In addition, many individual visitors were received. Considerable time has been allotted to the reception of visitors, and this policy has been considered worthwhile. Students of sanitary science have profited by plant demonstrations, and residents of Cambridge have acquired knowledge of details of the public water supply, a topic about which the average layman possesses little information.

Defense Program. In accordance with recommendations of the National Defense Council, measures have been instituted to safeguard the filtration plant, pumping station, and other department facilities against possible danger. The filter building is now closed to visitors, with the exception of those who have received written permits from the general superintendent. Flood lights illuminate the building and grounds at night, and a patrol schedule has been prepared for the watchmen.

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CORROSION CONTROL

A Critical Review of Recent Developments

BY EDWARD W. MOORE*

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At the time when the author first entered the ranks of the sanitary engineering profession, there were two schools of thought regarding corrosion control. One school held that the proper thing was to increase the pH value of the corrosive water by any convenient means, and the problem would then be solved. The other school, regarded as somewhat radical, although including some of the outstanding leaders in the field, contended that the thing really necessary was to treat the corrosive water in such a way that some sort of protective coating would be formed on the piping, and that within reasonable limits the pH value really mattered very little, so long as this objective was achieved. Credit for the first clear exposition of this theory should probably go to Baylis (1).

Lime. As the researches of Baylis, Clark, Hale, Hoover and many others tended increasingly to discredit the pH theory, soda ash as a corrective gave way to lime, particularly for softer waters. The objective of lime treatment or, under the new theory, of any of the common alkaline treating agents, was to deposit and maintain on the pipe a thin film of calcium carbonate or other insoluble substances, which would then protect the metal from corrosion. The dosages were at first determined by trial and error, coupled with observations of the condition of the system. The first method for predetermining the amount of chemical to be used for a given water was Baylis' marble test. At a later date the complete theory of the calcium carbonate deposition equilibrium was worked out by Langelier (2), and Enslow (3) developed his Continuous Stability Indicator. Meanwhile, it had become evident that, in practice, the deposition of calcium carbonate often failed to be extended to the ends of the system, and that increased dosages might merely increase the incrustation of that part

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of the system nearest the plant. A method of delaying the carbonate precipitation was obviously desirable.

Metaphosphates. At about this time the profession began to hear about the peculiar properties of certain of the phosphates, notably sodium hexametaphosphate and sodium pyrophosphate. These compounds had been known to chemists for a hundred years or more but had never been produced commercially. In 1929, the hexametaphosphate was made available and its property of inhibiting the precipitation of calcium compounds and other insoluble substances led to its extensive utilization in many fields. The use of this material to inhibit or delay precipitation of calcium carbonate, so that the advantages of treatment might be extended to the ends of the water system, was suggested by the work of Rosenstein (4) on irrigation waters and was further developed by Rice and Hatch (5). It was shown that incrustation of nearby parts of the water system could be avoided, while the slow reversion of the metaphosphate permitted gradual precipitation of carbonate in the remote parts of the system. This desirable result could be accomplished by dosages of the order of a few parts per million. Pyrophosphates have somewhat similar properties, but less seems to be known about them.

No sooner had the use of metaphosphate in conjunction with lime been accepted, than a further discovery was announced. It was claimed that the metaphosphate alone was capable of inhibiting corrosion and that the water did not have to be saturated with calcium carbonate. The use of metaphosphate in soft waters, without the addition of lime, and with no regard to the calcium carbonate equilibrium, was recommended and has been adopted in a number of places.

In its application in conjunction with lime, the metaphosphate was considered to perform its beneficial function by isolating the crystal nuclei of calcium carbonate as soon as formed, thus preventing further growth and deposition. It was also found that the metaphosphate tended not only to prevent the formation of new desposits but to remove those already existing. In either case, the action of the metaphosphate was not apparent until the surfaces had become saturated with it.

The action of the metaphosphate alone in inhibiting corrosion is reported to be due to its ability to form a film on the metal, probably by absorption. The thickness of the film may be only of molecular

dimensions. Another property that may account in part for the apparent inhibition of corrosion is the ability of the material to prevent subsequent formation, from the corrosion products, of visible precipitates such as iron oxides. While this is advantageous from the standpoint of water quality, it is not really a protection of the pipe from corrosion, in the sense of preventing solution of metal from the pipe. In spite of the significance of this question, little quantitative information is available on the relative importance of the two effects. If the inhibition of corrosion by metaphosphate is more apparent than real, then the appearance of the water delivered may be improved by its use, but the service life of the piping will not be greatly extended.

There are numerous reports in the literature of successful utilization of metaphosphates in conjunction with lime or other alkalies. East Rochester, N. Y., has used metaphosphate in conjunction with silicate since 1939. The silicate was discontinued recently. Freeport, L. I., experienced difficulty with incrustation when using lime only but found that metaphosphate reduced the deposits. Trax (6) has described a water system in western Pennsylvania in which the addition of soda ash was the only treatment practiced for many years. After the construction of a filtration plant, it was found necessary to substitute lime. Excessive deposits of calcium carbonate occurred within the plant but were cleared up by the use of metaphosphate. Many other cases could be cited, but these will suffice by way of illustration.

The use of metaphosphates alone on soft, corrosive waters is more recent, and consequently experience is more limited. At Wilkinsburg, Pa. (7), metaphosphates were originally applied to stabilize a lime-treated water during periods when manganese removal was necessary, but their use was continued as a corrosion-inhibiting measure when lime treatment was stopped. Effective corrosion control was apparently obtained at 0.5 p.p.m. However, certain difficulties were encountered, the principal one being the loosening of old deposits in the mains, with resulting temporary deterioration of water quality. This difficulty has been reported in other instances. There was some loss of metaphosphate on the filters, and clumps of calcium phosphate-cemented sand and anthrafilt grains were formed. The plant chemist expressed a belief that bacterial growth was stimulated by the metaphosphate, but the published results cannot be regarded as definite

proof of this contention. Gidley and Weston (8) have reported the use of metaphosphate at Fairhaven. Here, corrosion control was not the only objective, as it was also desired to prevent the deposition of iron already present in the water.

Silicates. The use of alkaline silicates for corrosion control also derives from the protective-film theory and dates back to 1922, when it was first suggested by Thresh (9) to prevent the solution of lead from lead pipe in England. The film formed on the pipe may be either a silica gel, or, more probably, an insoluble silicate of calcium or some other metal. The advocates of silicate treatment claim that control of dosage is simple enough to make the process available to small systems, that the hardness of the water is not increased by the treatment, and that good corrosion protection is obtained. Weston (10) has commented favorably on the alternate use of lime and silicate, as employed at Milford and Winchester. Hale (11), however, has stated that the protection is obtained at the cost of building up heavy incrustations.

Stericker (12) recommends a minimum concentration of 8 p.p.m. of silica as SiO_2 in the water, or 28 p.p.m. of the particular sodium silicate solution (Na_2O : 3.25 SiO_2) employed for most waters. Of course, such water would not be satisfactory for use in high-pressure boilers, for which silica tolerances are as low as 1 p.p.m. On the other hand, rayon plants, in which the least trace of iron in the process water is undesirable, have used as much as 45 p.p.m. of silica to curb solution of iron from piping and equipment.

At Eagles Mere, Pa., treatment of the soft, acid water to pH 8.6 with soda ash failed to reduce the iron content at the extreme ends of the system below 1.6 p.p.m. as Fe. Addition of silicate immediately reduced this figure to 0.7 p.p.m. and later to 0.3 p.p.m. A change to a more alkaline type of silicate solution resulted in an immediate sloughing of old deposits, raising the iron content of the water to 3.8 p.p.m.; within two days, however, the iron had dropped back to 0.35 p.p.m. This experience is reminiscent of the effects noted with metaphosphates, and suggests that caution should be exercised in introducing or altering anti-corrosion treatments of this type.

Many of the applications of silicate in municipal supplies have been made in conjunction with lime. The lime has usually been applied continuously and the silicate at intervals. The coating produced in

this way is hard, smooth, and dense, and the combination is regarded by many as sufficiently superior to lime alone to justify the additional expense.

Magno. In 1931, there was reported from Germany a new contact neutralizing medium, made by partial calcination of dolomite to $CaCO_3$ and MgO. This material is used in filter beds, much as limestone has been used in this country, and performs the same function: that of converting the carbon dioxide of the water to bicarbonate ions, associated with calcium and magnesium ions. On the basis of the calcium carbonate saturation theory, it would seem that this material should be no more, and probably less, effective than a limestone bed for straight corrosion control, since the magnesium dissolved from the material would increase the hardness of the water without greatly affecting its saturation equilibrium. However, it may be possible that the rate of solution of the material is somewhat greater, thus permitting a greater flow per unit volume of bed. Magno has also been used for deferrization and demanganization, and for these purposes the presence of the magnesium oxide probably has a special value. The war has cancelled plans for exporting the material to this country, and information regarding its application is meagre for the same reason.

Cathodic Protection. It may be worth while to include in this discussion an entirely different method of corrosion control, which has not yet been applied to the protection of the interior of pipes but has been quite successful in mitigating exterior soil corrosion of pipes and has proved to be of value for standpipes and similar structures. "Cathodic protection," so-called, consists essentially of setting up an artificial or applied potential or voltage difference between the metal and the corrosive medium such that the positive ions tend to flow from the medium to the metal rather than from the metal to the medium. Since corrosion requires that positive ions leave the metal, it is greatly diminished by the opposing potential. In addition, the electrolytic action appears to deposit a protective layer of variable composition on the surface of the metal.

The history of cathodic protection is obscure. Patents were taken out in Australia in 1911 and possibly in Germany in 1908. Protection was originally applied to long oil or gas lines from corrosion by soils. It was first used in a water supply system in 1928, when it was applied

to the Mokelumne Aqueduct to protect the pipe at points where the felt-wrapped bituminous covering had cracked. The last few years have seen the application of cathodic protection to steel standpipes. Although it is too early to draw definite conclusions, reports indicate that the method is highly successful in protecting standpipes against corrosion, and that the only practical difficulty seems to be the preservation of the anode system from destruction by ice movement.

The apparatus consists of a copper-oxide or selenium rectifier (or a motor-generator set in the earlier installation) delivering anywhere from 5 to 150 amperes of direct current at from 2 to 10 volts, depending on the size of the object to be protected. The negative pole of the rectifier is connected to the object to be protected, and the positive pole is connected to a suitable electrode system. In standpipes, the electrode system consists of one or more stainless steel or carbon electrodes mounted below the minimum water level inside the tank.

Few reports are as yet available on the operation of standpipe installations, but it has been reported in one instance that the cost of the installation plus current for one year was materially less than the cost of a single painting and that the standpipe was in excellent condition at the end of a year. A thin gray deposit was noted on the interior walls.

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HYDRAULIC INVESTIGATION OF WATER DISTRIBUTION SYSTEMS IN FIELD AND OFFICE.

BY GORDON M. FAIR*

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No component of public water supplies stands in greater need of systematic and repeated study than the network of mains and laterals that conveys water through the streets of the community to household and industry and provides the water necessary for other municipal purposes, especially for fire fighting. In respect to first cost and upkeep, the distribution system is generally the most costly portion of the water works. Because of growth or shifts in population and industry, the demands made upon the system are constantly undergoing change. As a result, the network itself must be extended, reinforced, or otherwise modified at short intervals of time to meet new needs. Small wonder that the administration, maintenance and reconstruction of distribution systems occupy the bulk of the time and effort of water works superintendents and their staffs.

In these circumstances, it is unfortunate that the hydraulics of distribution systems appear to be generally more complicated and less amenable to rigorous analysis than the hydraulics of most other common components of water works. Considerations of service, safety, and economy, however, require that the hydraulic performance of distribution systems, under all conceivable conditions, be known or calculated with sufficient reliability to be of value in planning and operation. It is the purpose of this paper to sketch briefly the background against which the hydraulics of distribution systems must be viewed or tested and to present those methods of investigation—many of them old, but some of them new—that may be found of assistance in the analysis of the hydraulic characteristics of distribution systems in the field and in the office.

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Types of Distribution Systems

Street plan, topography, and location of supply works and distribution storage establish the type of distribution system and the character of flow through it.

Grid-iron and Branching Systems. Depending upon street plan, the distribution system forms a network of pipes in which two patterns predominate: (a) a branching pattern on the outskirts of the community, in which ribbon development follows the primary arteries of roads and streets (Fig. 1a), and (b) a grid-iron pattern within the

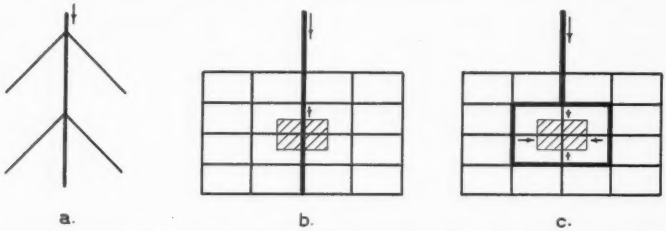


FIG. 1—PLANS OF BRANCHING SYSTEM AND GRID-IRON SYSTEM WITH CENTRAL AND LOOPED FEEDERS—IDEALIZED—MINOR SUBDIVISIONS NOT SHOWN.

a. Branching pattern.
b. Grid-iron pattern with central feeder. Flow to high-value district (cross-hatched) through one feeder.
c. Grid-iron pattern with looped feeder. Flow to high-value district (cross-hatched) through two or more, feeders.

built-up portion of the community in which streets criss-cross and pipes are interconnected (Fig. 1b and c). Hydraulically, the grid-iron system possesses the advantage of carrying water to any spot from more than one direction; the branching system the disadvantage of dead ends. The carrying capacity of the network, or reticulation, system is strengthened by providing, in place of a central feeder, a loop or belt of feeder pipes that permits drawing water to the congested, or high-value, district from at least two directions, thereby more or less doubling the delivery of the grid (Fig. 1c).

Single and Dual Mains. As outlined in Fig. 2, a distribution system of single mains, customarily laid on the north and east sides of streets for protection against freezing, is provided with valves as follows: three at crosses, two at tees, and one on the single hydrant branches. In the dual-main system, service headers are added on the south and

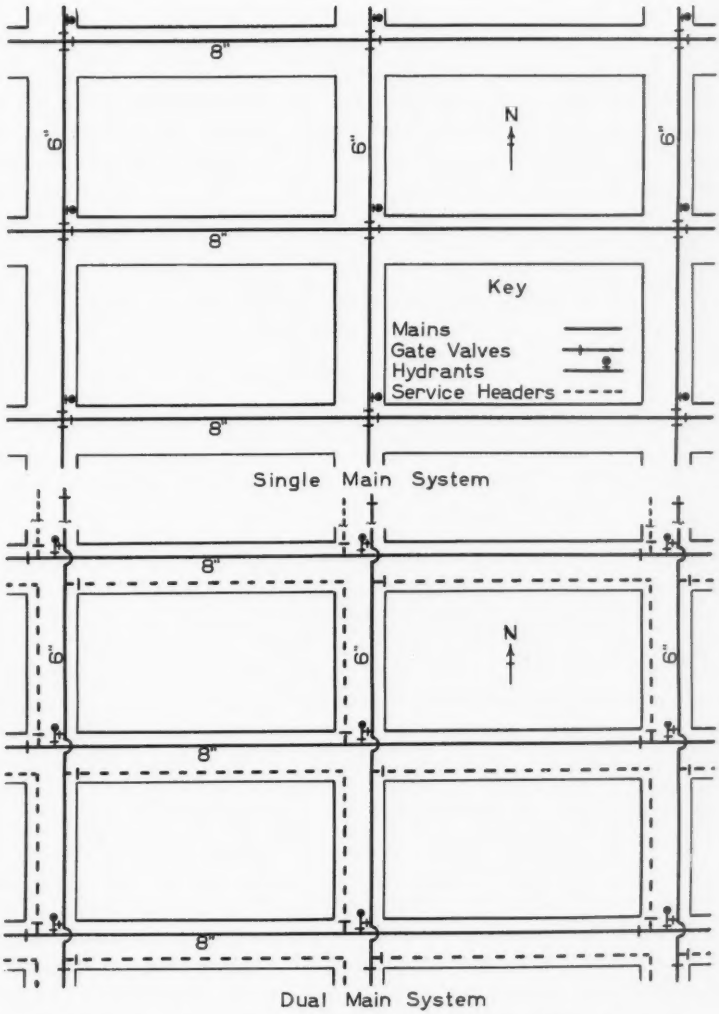


FIG. 2—PLANS OF GRID-IRON SYSTEM OF SINGLE AND DUAL MAINS—IDEALIZED.
Dual-Main System after Ballou.

west sides of streets and piping is generally placed beneath the sidewalks. The scheme suggested by Ballou* includes the following valves: one on each main at intervals of two blocks, one at the junction of each service header with its main, and one on each of two branches to the hydrants. The operations necessitated and conditions of service created by breakdowns are summarized in the following schedule.

Breakdown of	*Single-main System*	*Dual-main System*
1. hydrant	Close one hydrant valve.	Close two hydrant valves.
2. east-west main	Close four valves on main. This dead-ends supply to one hydrant and cuts out intermediate taps; it also dead-ends supply to eastern intersection.	Close two valves on main and one valve on each of the connected hydrants and service headers. Hydrants and service headers remain in commission; house service taps on main are placed out of commission; service headers are dead-ended.
3. north-south main	Close two valves on main. One hydrant and intermediate house taps are placed out of commission.	Same as for east-west main.

Hydraulically, the differences between single and dual mains are more or less local in character. The service headers of dual systems do not contribute to flow outside of their own block. Breaks in mains, however, fail to impair the usefulness of hydrants and to dead-end mains. The value of dual mains in the maintenance of water supply during destructive air raids or other enemy action is obvious.

High and Low Service. Sections of a community that lie at too high an elevation to receive water at adequate pressure from the principal, or low-service, works are generally incorporated in a separate distribution system possessing independent piping and service storage. This system is normally fed by pumps that take suction from the main supply and boost its pressure by the requisite amount. For areas of widely varying elevation, intermediate services may be

*This Journal, **52** (1938), 81.

required. The different service systems are commonly interconnected for emergency use. Hydraulic analysis must take cognizance of the independence as well as the interdependence of such systems. High-service draft from low-service arteries deserves particular recognition. Since high-service areas are commonly small in extent, the support that the low service may expect to receive from high-service storage, in case of a breakdown in the main supply, is generally disappointing.

High-pressure Fire Supply. The congested, central portion, or high-value district, of large cities is suitably protected by a separate system of pipes and hydrants that is capable of delivering large volumes of water under high pressure for fire-fighting purposes. To avoid the hazards of cross-connected supplies, this auxiliary fire service normally takes water from the public supply and raises its pressure by booster pumps whenever the alarm is given to do so. For use in extreme emergency, rigorously protected connections may be established to an independent source of water: river, lake or tidal estuary. Hydraulically, a system of this kind occupies much the same status as a high-service supply. It is independent in its larger functions of water distribution but does draw upon the general water works for its supply. Such dependence may be greatly reduced by the provision of adequate storage from which the fire supply is fed.

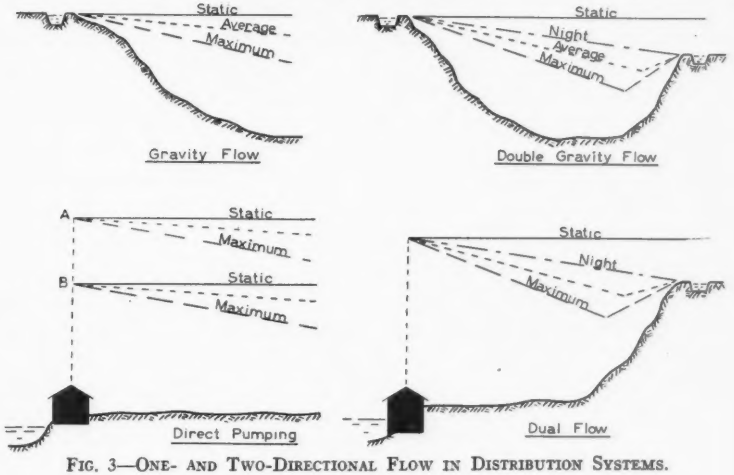


FIG. 3—ONE- AND TWO-DIRECTIONAL FLOW IN DISTRIBUTION SYSTEMS.

One- and Two-directional Flow. Four general systems of flow are illustrated in Figure 3. The hydraulic gradients of these systems and the residual pressures within the areas served, together with the volume of distribution storage, determine the sizes of pipe within the network. It is plain that flow from opposite directions will increase the capacity of the distribution system. Two-directional flow in the main arteries themselves is made possible by conducting the supply from pumps, or from gravity supply, or service, reservoirs, to opposite ends of the feeder system or through it to elevated storage in a reservoir, tank, or standpipe situated towards the far end of the area of greatest water demand. Selection of location and size of distribution storage depends upon topography and water needs as explained later in this paper.

Required Capacity and Pressure of Distribution Systems

The capacity of distribution systems is dictated by domestic, industrial, and other normal water use and by the *standby* or *ready-to-serve* requirements for fire fighting. The capacity to serve cannot be expressed, however, merely in terms of the rate at which water can be drawn. The pressure at which this water is made available is equally important, because the water must rise to the upper stories of buildings of normal height and flow from hydrants, directly or through pumpers, to form the fire streams issuing from the nozzles of fire hose that is long enough to reach the scene of conflagration.

Capacity. If there was no fire hazard, the hydraulic capacity of the distribution system would have to equal the maximum demand for domestic, industrial, and other general uses. For absolute safety, the fire demand would be added to this figure. Ordinarily this is not done: (1) because it would be most unusual for the maximum draft to coincide with a serious conflagration, and (2) because systems are dimensioned for the future and new construction is generally completed, in a reasonably foresighted community, before the designed capacity of the original system has been reached. In the absence of unusual hazards, in the way of inflammable structures or storage of inflammable raw or manufactured materials, a per capita draft of 40 g.p.d. in excess of the average annual consumption* may ordinarily

*The average annual consumption varies normally from about 65 g.p.d. per capita in fully metered communities to about 150 g.p.d. per capita in the absence of metering. Industrial demands may increase these values appreciably.

be assumed to coincide with the requisite fire flow. In the presence of unusual hazards, higher allowances should be made. Determination of the requisite capacity of the distribution system within different areas of the community is aided greatly if the municipality has been "zoned," and the ordinances are well drawn and regulate both the *use* to which properties may be put in different zones and the *bulk* that the buildings to be erected may occupy.

The rate at which water should be made available to extinguish fires has been studied in particular by the National Board of Fire Underwriters. The broad experience of this organization has led it to formulate certain general requirements from which the water works engineer should depart only after careful study and for good and sufficient reasons. These requirements may be outlined as follows:

1. Within the central, congested, or high-value district, the rate, G in g.p.m., at which water should be made available in order to check a serious conflagration varies with the population, P in thousands, in accordance with the following relationship for communities of 200,000 people or less:

$$G = 1{,}020\sqrt{P}\,(1 - 0.01\sqrt{P})$$

TABLE 1—REQUIRED FIRE FLOW, FIRE RESERVE, AND HYDRANT SPACING. Recommended by the National Board of Fire Underwriters.

Population	Fire Flow g.p.m.	Fire Flow m.g.d.	Fire Reserve m.g.	Area per Hydrant sq. ft. Engine Streams	Area per Hydrant sq. ft. Hydrant Streams
1,000	1,000	1.5	0.3	120,000	100,000
2,000	1,500	2.0	0.4	———	90,000
4,000	2,000	3.0	1.2	110,000	85,000
6,000	2,500	3.5	1.5	———	78,000
10,000	3,000	4.5	1.9	100,000	70,000
13,000	3,500	5.0	2.1	———	———
17,000	4,000	6.0	2.4	90,000	55,000
22,000	4,500	6.5	2.7	———	———
28,000	5,000	7.5	3.0	85,000	40,000†
40,000	6,000	8.5	3.6	80,000	———
60,000	7,000	10.5	4.4	70,000	———
80,000	8,000	12.0	5.0	60,000	———
100,000	9,000	13.0	5.5	55,000	———
125,000	10,000	14.5	6.1	48,000	———
150,000	11,000	16.0	6.7	43,000	———
200,000*	12,000	18.5	7.6	40,000	———

*For populations over 200,000 and local concentration of streams, see outline of National Board requirements.

†For fire flows of 5,000 g.p.m. and over.

Table 1 is based upon this equation and shows the relatively large stand-by capacity needed.

2. The central portion of cities with populations in excess of 200,000 requires capacities of 12,000 g.p.m. and from 2,000 to 8,000 g.p.m. in addition for a second fire.

3. For residential districts, the required fire flow depends upon the character and congestion of the buildings about as follows:

a. Small, low buildings—1/3 of lots in block built upon ..... 500 g.p.m.
b. Larger or higher buildings ..... 1,000 g.p.m.
c. Buildings approaching dimensions of hotels or high-value residences ..... 1,500 to 3,000 g.p.m.
d. Three-story buildings—densely built up sections up to 6,000 g.p.m.

4. In the high-value district, it must be possible to concentrate 2/3 of the estimated flow on one block or one very large building.

5. In compact residential areas, ¼ to ½ the estimated flow must be within reach of one block or one very large building.

6. For detached buildings it must be possible to muster 500 to 750 g.p.m.

Pressure. For normal drafts, the pressure at the street line must be at least 20 lb. per sq. in. (46 ft. of water) in order to raise water three stories and overcome the frictional resistance of the house-distribution system; but 40 lb. per sq. in. is a more desirable pressure. Business blocks are supplied better at pressures of 60 to 75 lb. per sq. in. Fire demand is commonly gaged in terms of the standard fire stream: 250 g.p.m. issuing from a 1⅛-in. nozzle and requiring a pressure at the base of the tip of 45 lb. per sq. in. When this amount of water flows through 2½-in. rubber-lined fire hose, the frictional resistance to flow is approximately 15 lb. per sq. in. per 100 ft. of hose. Adding to this hydraulic loss the hydrant resistance and the required nozzle pressure of 45 lb. per sq. in. gives the following pressure needs at the hydrant according to Freeman:

Length of hose:	100	200	300	400	500	600 ft.
Required pressure:	63	77	92	106	121	135 lb. per sq. in

Since hydrants are normally planned to protect areas within a radius

of 200 ft., it is evident that for direct attachment of fire hose to hydrants (hydrant streams) the required residual pressure at the hydrant must be about 75 lb. per sq. in. To maintain this pressure at times of fire, normal pressures must approach 100 lb. per sq. in. Objections to pressures as high as this are increased leakage and waste of water, approximately in proportion to the square root of the pressure. The minimum pressure for hydrant streams is commonly set at 50 lb. per sq. in. Such streams, however, will not approach standard magnitudes after passing through hose as much as 50 ft. in length.

Modern motor pumpers will deliver up to 1,500 g.p.m. at adequate pressures, and many larger cities use single streams discharging as much as 1,000 g.p.m. from a 2-in. nozzle. In order that domestic draft may be maintained and the system safeguarded against pollution by seepage water and failure under a vacuum, fire engines are not expected to pull down the pressure in the mains below 20 lb. per sq. in. For large hydrant outlets, this limit is sometimes lowered to 10 lb. per sq. in. In a sense, modern fire-fighting equipment has reduced the necessity for pressures much in excess of 60 lb. per sq. in. except in small communities that cannot afford a full-time, well-equipped fire department. The additional pressure drop through the system made available by the use of pumpers increases the fire-fighting capacity in the ratio of $\sqrt{P-20} \div \sqrt{P-75}$, where P is the normal dynamic pressure of the system.

Structural Components of Distributing Systems

The basic elements of the reticulation system are its pipes, gates and hydrants. The dimensioning and spacing of these components rests upon a background of experience that is normally sufficiently precise in its minimum standards to permit roughing in all but the main arteries and feeders of most distribution systems. Common standards include the following recommendations:

Pipes.

Smallest pipes in gridiron	6-in.
Smallest branching pipes (dead ends)	8-in.
Largest spacing of 6-in. grid (8-in. pipe used beyond this value)	600 ft.
Smallest pipes in high-value district	8-in.

	Smallest pipes on principal streets in central district	12-in.
	Largest spacing of supply mains or feeders	2,000 ft.
Gates.	Spacing in single- and dual-main systems	See preceding discussion
	Largest spacing on long branches	800 ft.
	Largest spacing in high-value district	500 ft.
Hydrants.	Areas protected by hydrants	See Table 1
	Largest spacing when fireflow exceeds 5,000 g.p.m.	200 ft.
	Largest spacing when fireflow is as low as 1,000 g.p.m.	300 ft.

Pipe sizes in excess of the minimum are determined by the occupancy of the properties along the lines, whether residential, commercial, or industrial, and by the water uses of each together with the fire risks involved.

The "hydrant areas" shown in Table 1 are based upon a single fire stream being effective within a radius of 200 ft. from the hydrant. The area of the resulting circle is 120,000 sq. ft., and in order to attack a fire from all sides, or at least from two hydrants, a minimum of four streams (1,000 g.p.m.) must be brought to play upon this area. As communities increase in size, buildings grow in bulk and the area served by each hydrant must be reduced. Hydrant streams tax the distribution system more than do engine streams and have less leeway when pressures drop. Allowance is made for this by reducing the size of the area to be served by each hydrant. In all these standards, experience with conflagration underlies the recommended values.

Field Studies of Distribution Systems

The hydraulic performance of existing distribution systems is determined most directly and expeditiously by pressure surveys and hydrant-flow tests. There is no limit to the extent of such tests. They should cover all typical portions of the community: the high-value district, residential neighborhoods of different kinds, industrial

areas, the outskirts, and high-service zones. If need be, they can be extended into every block. The results obtained will establish available pressures and flows and existing deficiencies. These can then be made the basis of hydraulic calculations for extensions, reinforcements, and new grid-iron layouts. Subsequent tests can be conducted to show how nearly the desired changes have been accomplished.

Pressure Surveys. These yield the most rudimentary information about the network. If they are conducted both at night (minimum flow) and during the day (normal demand), they will indicate the hydraulic efficiency of the system in meeting common requirements. The information presented, however, is not sufficient to establish the probable behavior of the system under conditions of stress such as are produced by a serious conflagration.

Hydrant-flow Tests. As commonly performed, hydrant-flow tests include (1) observation of the pressure at a centrally situated hydrant during the conduct of the test and (2) measurement of the flow from a group of neighboring hydrants by means of hydrant Pitot tubes that record the velocity heads in the jets issuing from the hydrants. To be significant, the following precautions should be observed:

1. The hydrants tested should form a group such as might be called into play in fighting a serious fire in the district under study.

2. Water should be drawn at a sufficient rate to create a drop in pressure so great that its value is not measurably affected by normal fluctuations in draft within the system.

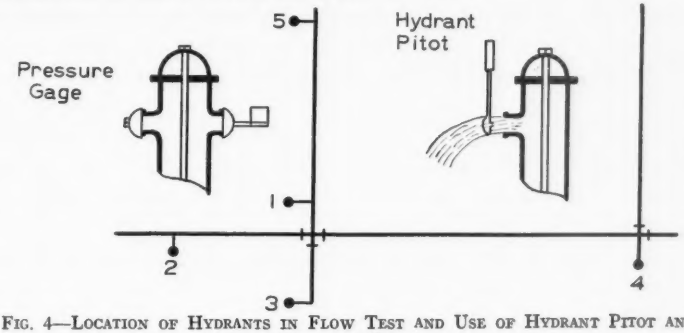


FIG. 4—LOCATION OF HYDRANTS IN FLOW TEST AND USE OF HYDRANT PITOT AND PRESSURE GAGE.
See Table 2 and Fig. 5.

3. The time of test should coincide with drafts (domestic, industrial, etc.) in the remainder of the system, reasonably close to expected values.

The requirements of the National Board of Fire Underwriters that have already been outlined are valuable aids in planning hydrant-flow tests. The layout of pipes and hydrants in a typical flow test is shown in Figure 4 and the observed values are summarized in Table 2.

TABLE 2—RECORD OF A TYPICAL HYDRANT-FLOW TEST.
All pressures are expressed in lb. per sq. in.

Conditions of Test	Observed Pressure at No. 1	Observed Discharge Pressure (p) (velocity head)	Calculated Flow (Q) g.p.m.	Remarks
All hydrants closed	74	—	—	All hydrant outlets are
No. 2 opened, 1 outlet	—	13.2	610	2½ in. in diameter.
No. 3 opened, 2 outlets	—	9.6	2 x 520	
No. 4 opened, 1 outlet	—	16.8	690	Total Q = 2,980 g.p.m.
No. 5 opened, 1 outlet	46	14.5	640	Calculated engine
All hydrants closed	74	—	—	streams 4,200 g.p.m.

This table is more or less self-explanatory. The initial and residual pressure was read from a Bourdon gage at hydrant 1, and hydrants 2, 3, 4, and 5 were opened in quick succession and their rate of discharge measured simultaneously by means of hydrant Pitots. A test such as this does not consume more than five minutes if it is conducted by a well-trained crew.

Hydrant-flow Calculations. The necessary calculations may be outlined as follows:

1. For outlets of diameter d in., the discharge Q in g.p.m. is: $Q = 29.83\ cd^2\sqrt{p}$,* where p is the Pitot reading in lb. per sq. in. and c is the coefficient of hydrant discharge. For smooth, well-rounded, 2½-in. outlets $c = 0.9$ and $Q = 168.2\ \sqrt{p}$.

2. The total discharge is 2,980 g.p.m. for a pressure drop of 71 — 46 = 28 lb. per sq. in.

*Since $Q = c\ a\ v$ where c is the coefficient of hydrant discharge, a is the area of the hydrant outlet and v is the velocity of discharge ($2.308\ p = v^2 \div 2g$). Here Q is measured in c.f.s., a in sq. ft., and v in ft. per sec. The value of c varies from 0.9 for well-rounded, smooth outlets to 0.7 for sharp outlets projecting into the barrel.

3. For engine streams the pressure drop is 74 — 20 = 54 lb. per sq. in. and the approximate, expected discharge $Q_2 = Q_1\sqrt{P_2} \div P_1 = 2{,}980\sqrt{54} \div 28 = 4{,}150$ g.p.m.

4. Since most of the loss occurs in the piping rather than the hydrant outlet, the expected discharge is calculated more closely from the Williams-Hazen Formula, or $Q_2 = 2{,}980\left(\frac{54}{28}\right)^{0.54} = 4{,}250$ g.p.m. As shown in the appendix to this paper, this value can be read directly from the Williams-Hazen diagram (see Fig. 11e).

5. If the required fireflow in this district is estimated to be 6,000 g.p.m., the deficiency is 6,000 — 4,200 = 1,800 g.p.m. and must be supplied by the addition of suitable piping.

The pressure-discharge relations established in this test are illus-

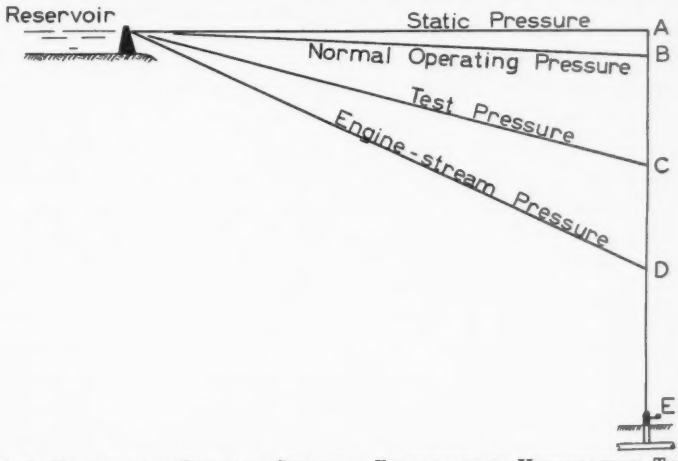


FIG. 5—PRESSURE AND DISCHARGE RELATIONS ESTABLISHED BY HYDRANT-FLOW TEST. See Fig. 4 and Table 2.

A. Static water table.
B. No hydrant discharge. Pressure = 74 lb. per sq. in.
Pressure drop P_0 due to coincident draft Q_0.
C. Hydrant discharge. Pressure = 46 lb. per sq. in.
Pressure drop P_1 = 74 — 46 = 28 lb. Accompanies discharge of Q_1 = 2,980 g.p.m.
D. Engine streams. Pressure = 20 lb. per sq. in.
Pressure drop P_2 = 74 — 20 = 54 lb. Accompanies discharge of Q_2 = 4,200 g.p.m.
E. Hydrant No. 1 recording residual pressure of hydrant group shown in Fig. 4.

trated in Fig. 5. If the true static pressure is known, a more exact calculation than that here proposed is possible, although the results seldom justify the additional labor involved. If a pressure gage is inserted in a hydrant in exact juxtaposition to the hydrant outlet that is to be opened, the pressure recorded will equal the discharge pressure. Difficulty of centering such a gage generally makes the use of a hydrant Pitot more satisfactory and reliable.

Hydrant tests of individual hydrants are of limited value only and may be quite misleading.

Office Studies of Distribution Systems

No matter how energetically the study of distribution systems is pursued in the field, hydraulic investigations of extensions, reinforcements and new networks can be brought to satisfactory conclusion only in the office. The necessary analysis presupposes some familiarity with certain processes of hydraulic computation. While, at first glance, some of these processes may appear to be complicated, closer scrutiny will show that the best of them can be so systematized as to make their application a matter of simple arithmetic calculations in addition to the use of a pipe-flow table or a diagram such as the Williams and Hazen diagram appended to this paper.

Three methods of analysis will be found to be particularly useful. These will be defined as: (1) the method of sections; (2) the Hardy Cross method; and (3) the method of equivalent pipes which may be applied alone or in conjunction with the Hardy Cross method.

Method of Sections. This is an approximate method, simple in concept and application, and widely useful provided that its limitations are clearly acknowledged and allowed for. To the author's knowledge, the method of sections was employed by Allen Hazen as a quick check of distribution systems. So far as the author knows, however, it has never been published, certainly not in the form in which it will be presented here. Pardoe's method* is somewhat like it but is more involved. Of similar concept, too, is the circle method described in most textbooks on water supply but confined in its application to the cutting by a circle of the system of pipes tributary to a central fire hydrant or group of hydrants.

*Engineering News-Record, 93 (1924), 516.

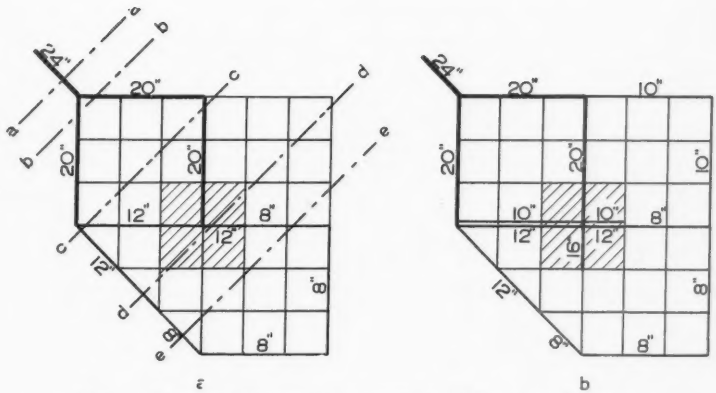


FIG. 6—PLAN OF NETWORK ANALYZED BY METHOD OF SECTIONS. See Table 3.

a. Existing System.
b. Recommended System.

When pipe size is not indicated, the pipe diameter is 6 in. The cross-hatched area is the high-value district.

Use of the method of sections is illustrated in Figure 6 and Table 3. The various steps involved may be outlined as follows:

1. Cut the network by a series of lines, not necessarily straight or regularly spaced but chosen with due regard to the varying sequence of pipe sizes and character of district. A first series of lines may well be chosen so as to cut the distribution piping substantially at right angles to the general direction of flow, *i.e.* perpendicular to a line drawn from the supply conduit to the high-value district (see Fig. 6). Further series may be oriented in some other critical direction; for example, horizontally and vertically in Fig. 6. If there is more than one supply conduit, the sections may be curved to intercept the flow from each.

2. Estimate the water that must be supplied to the areas *beyond* each section. Estimates are based on a knowledge of the population density and the general character of the zone: residential, commercial and industrial. The water requirements comprise (a) the normal, coincident draft, here called the domestic draft, and (b) the fire demand (see Table 1). Whereas domestic use decreases progressively from section to section, as population or industry is left be-

TABLE 3—ANALYSIS OF A NETWORK (FIGURE 6) BY THE METHOD OF SECTIONS. The hydraulic gradient available within the network proper is estimated to lie close to 2 ft. per 1,000 and the value of C in the Williams and Hazen formula is taken to be 100. The domestic (coincident) draft is assumed to be 140 g.p.d. per capita. The fire demand is taken from Table 1.

1. Section *a-a*. Population 16,000			
a. Demands:		Domestic	= 2.2 m.g.d.
		Fire	= 5.6 m.g.d.
		Total	= 7.8 m.g.d.
b. Existing pipes: 1, 24-in.		Capacity	= 6.0 m.g.d.
c.		Deficiency	= 1.8 m.g.d.
d. If no pipes are added, the 24-in. pipe must carry 7.8 m.g.d. This it will do with a loss of head of 3.2 ft. per 1,000 at a velocity of 3.8 ft. per sec. (See Williams and Hazen diagram in Appendix.)			
2. Section *b-b*. Population and flow as in *a-a*.			
a.		Total demand	= 7.8 m.g.d.
b. Existing pipes: 2, 20-in.		Capacity	= 7.4 m.g.d.
c.		Deficiency	= 0.4 m.g.d.
d. If no pipes are added, existing pipes will carry 7.8 m. g. d. with a loss of head of 2.2 ft. per 1,000 at a velocity of 2.8 ft. per sec.			
3. Section *c-c*. Population 14,000			
a. Demands:		Domestic	= 2.0 m.g.d.
		Fire	= 5.6 m.g.d.
		Total	= 7.6 m.g.d.
b. Existing pipes:	1, 20-in.	Capacity	= 3.7 m.g.d.
	2, 12-in.	Capacity	= 2.0 m.g.d.
	5, 6-in.	Capacity	= 0.8 m.g.d.
		Total	= 6.5 m.g.d.
c.		Deficiency	= 1.1 m.g.d.
d. Pipes added:	2, 10-in.	Capacity	= 1.2 m.g.d.
Pipes removed:	1, 6-in.	Capacity	= 0.2 m.g.d.
		Net added capacity	= 1.0 m.g.d.
e.		Reinforced capacity	= 7.5 m.g.d.
The reinforced system (equivalent pipe* 26.0 in.) will carry 7.6 m.g.d. with a loss of head of 2.1 ft. per 1,000 at a velocity of 3.2 ft. per sec.			
4. Section *d-d*. Population 8,000			
a. Demands:		Domestic	= 1.1 m.g.d.
		Fire	= 5.6 m.g.d.
		Total	= 6.7 m.g.d.
b. Existing pipes:	2, 12 in.	Capacity	= 2.0 m.g.d.
	8, 6-in.	Capacity	= 1.3 m.g.d.
		Total	= 3.3 m.g.d.
c.		Deficiency	= 3.4 m.g.d.

*The equivalent pipe is one that will carry 7.5 m.g.d. on a hydraulic gradient of 2 ft. per 1,000.

Table 3 (continued)

d.	Pipes added: 1, 16-in.	Capacity	= 2.1 m.g.d.
	2, 10-in.	Capacity	= 1.2 m.g.d.
	Pipes removed: 1, 6-in.	Capacity	= 0.2 m.g.d.
		Net added capacity	= 3.1 m.g.d.
e.		Reinforced capacity	= 6.4 m.g.d.

The reinforced system (equivalent pipe = 24.6 in.) will carry 6.7 m.g.d. with a loss of head of 2.2 ft. per 1,000 at a velocity of 3.1 ft. per sec.

5. Section *e-e*. Population 3,000

a.	Demands:	Domestic	= 0.5 m.g.d.
		Fire	= 1.5 m.g.d.
		Total	= 2.0 m.g.d.
b.	Existing pipes: 2, 8-in.	Capacity	= 0.7 m.g.d.
	6, 6-in.	Capacity	= 0.9 m.g.d.
		Total	= 1.6 m.g.d.
c.		Deficiency	= 0.4 m.g.d.
d.	Pipes added: 1, 10-in.	Capacity	= 0.6 m.g.d.
	Pipes removed: 1, 6-in.	Capacity	= 0.2 m.g.d.
		Net added capacity	= 0.4 m.g.d.
e.		Reinforced capacity	= 2.0 m.g.d.

The reinforced system (equivalent pipe = 15.8 in.) will carry 2 m.g.d. with a loss of head of 2.0 ft. per 1,000 at a velocity of 2.3 ft. per sec.

hind, the fire demand remains the same until the high-value district has been passed; then it drops to the figure that applies to the type of outskirt area encountered.

3. Estimate the capacity of the distribution system at each section across the piping. To do this:

a. Tabulate the number of pipes of each size cut. Count only those pipes that deliver water in the general direction of flow that is being studied.

b. Determine the average available hydraulic gradient or frictional resistance. This depends (1) upon the pressure to be maintained in the system and (2) upon the allowable pipe velocity.*

*University of Illinois, Bulletin 286, 1936.

*If, for example, in a level region the distance from the junction of the supply conduit with the network to the end of the high-value district is 25,000 ft. and the pressure at the junction is 70 lb. per sq. in., the pressure drop available for engine streams is (70 — 20) = 50 lb. per sq. in. or 116 ft. and the resulting hydraulic gradient becomes 116 ÷ 25 = 4.6 ft. per 1,000. If the required capacity of this system is 17 m.g.d., the piping must have a diameter equivalent to a 30-in. line (see Williams-Hazen diagram in Appendix) and the pipe velocity will be 5.4 ft. per sec. To reduce this velocity below 4 ft. per sec., it would be necessary to add pipes to the system. If the reinforced network were equivalent to a 36-in. line, the velocity would be reduced to 3.7 ft. per sec. This would lower the hydraulic gradient to 1.9 ft. per 1,000.

Hydraulic gradients lie, ordinarily, between 1 and 3 ft. per 1,000, and velocities range from 2 to 4 ft. per sec.

4. On the basis of the available, or desirable, hydraulic gradient, determine the capacity of the existing pipes and sum them to obtain the total capacity.

5. Calculate the deficiency as the difference between the required and the existing capacity.

6. On the basis of the available, or desirable, hydraulic gradient, select the pipes to be added to the system in order to offset the deficiency. General familiarity with the community as well as studies of the network plan will aid judgment. Removal of existing small pipes to make way for larger mains must be taken into account.

7. Determine the size of pipe equivalent to the reinforced system and calculate the velocity of flow through the system. Excessive velocities and the dangerous water hammer that may accompany them should be avoided, if necessary by lowering the hydraulic gradient actually called into play.

8. Check important pressure requirements against the plan developed for the network.

Performance of the necessary calculations is outlined in Table 3 for the network shown in Figure 6. The method of sections is particularly useful (1) in the study of large and complicated distribution systems; (2) as a check upon other methods of hydraulic analysis; and (3) as a basis for further investigation of the system by more exact calculations.

Hardy Cross Method. This is a method by which systematic corrections are applied to an initial set of assumed flows until the network is balanced hydraulically. The mathematical derivation of the author's adaptation of the principles first enunciated by Hardy Cross* is presented in the appendix to this paper.

The basic equation for the flow correction is:

$$q = \frac{\text{Sum } H}{1.85 \text{ Sum } \frac{H}{Q}}$$

where Q = the flow assumed for any one pipe in the network; q = the flow by which Q must be corrected in order to reach a closer hydraulic balance of the network; H = loss of head through any one pipe in the

network for a flow of Q as determined by the Williams and Hazen formula for the flow of water through pipes; and 1.85 = a constant from the Williams and Hazen formula.

The systematic application of this equation permits the solution of complex networks by the use of a table or diagram of the Williams and Hazen formula (see Plate) and by the simple arithmetic processes of addition, subtraction, multiplication and division.

The correction q is only approximate. After it has been applied once to the assumed flows, the network will be more nearly in balance than it was at the beginning, and the process of correction can then be repeated as often as is necessary to perfect the balancing operations. The work involved is straightforward but is greatly facilitated by a satisfactory scheme of bookkeeping such as the one outlined in Table 4 for the network sketched in Fig. 7. This calculating system,

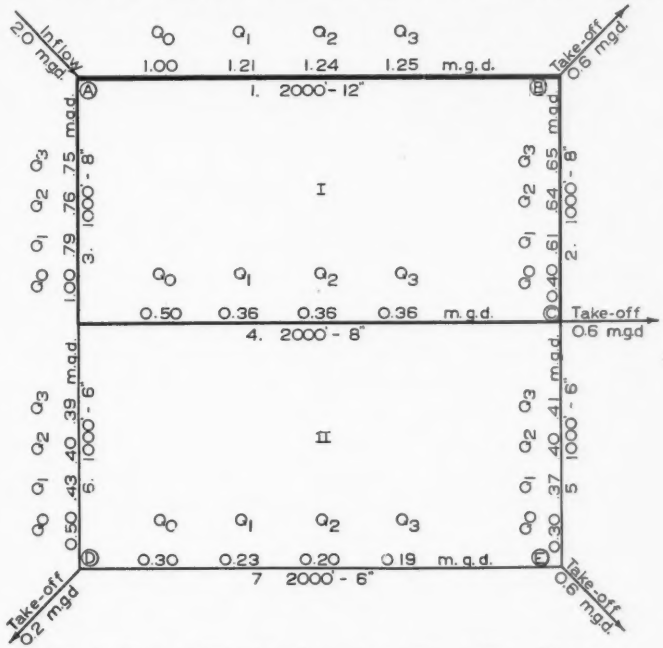


FIG. 7—PLAN OF NETWORK ANALYZED BY HARDY CROSS METHOD. See Table 4.

TABLE 4—ANALYSIS OF A NETWORK (FIGURE 7) BY THE HARDY CROSS METHOD

Circuit No. (1)	Pipe No. (2)	Length ft. (3)	Diam. in. (4)	Q_0 m.g.d. (5)	s_0 % (6)	H_0 ft. (7)	$\frac{H_0}{Q_0}$ (8)	q_0 (9)		Q_1 m.g.d. (10)	s_1 % (11)	H_1 ft. (12)	$\frac{H_1}{Q_1}$ (13)	q_1 (14)	
						Assumed Flow						First Correction			
I	1	2,000	12	+1.0	2.1	+ 4.2	4.2	+0.21		+1.21	3.0	+ 6.0	5.0	+0.03	
	2	1,000	8	+0.4	2.8	+ 2.8	7.0	+0.21		+0.61	6.1	+ 6.1	10.0	+0.03	
	3	1,000	8	—1.0	15.1	—15.1	15.1	+0.21#		—0.79	9.8	— 9.8	12.4	+0.03#	
	4*	2,000	8	—0.5	4.2	— 8.4	16.8	+0.21	—0.07	—0.36	2.3	— 4.6	12.8	+0.03	—0.03
						—16.5 ÷ 43.1 × 1.85 = —0.21						—2.3 ÷ 40.2 × 1.85 = —0.03			
II	4**	2,000	8	+0.5	4.2	+ 8.4	16.8	+0.07	—0.21	+0.36	2.3	+ 4.6	12.8	+0.03	—0.03
	5	1,000	6	+0.3	6.6	+ 6.6	22.0	+0.07		+0.37	9.8	+ 9.8	26.5	+0.03	
	6	1,000	6	—0.5	16.9	—16.9	33.8	+0.07#		—0.43	12.9	—12.9	30.0	+0.03#	
	7	2,000	6	—0.3	6.6	—13.2	44.0	+0.07		—0.23	4.1	— 8.2	35.6	+0.03	
						—15.1 ÷ 116.6 × 1.85 = —0.07						—6.7 ÷ 104.9 × 1.85 = —0.03			

Circuit No. (1)	Pipe No. (2)	Length ft. (3)	Diam. in. (4)	Q_2 m.g.d. (15)	s_2 % (16)	H_2 ft. (17)	$\frac{H_2}{Q_2}$ (18)	q_2 (19)		Q_3 m.g.d. (20)	s_3 % (21)	H_3 ft. (22)	Loss of Head A-E (23)
						Second Correction						Result	
I	1	2,000	12	+1.24	3.1	+ 6.2	5.0	+0.01		+1.25	3.2	+ 6.4	1. Via Pipes
	2	1,000	8	+0.64	6.6	+ 6.6	10.3	+0.01		+0.65	6.8	+ 6.8	1, 2, 5
	3	1,000	8	—0.76	9.1	— 9.1	12.0	+0.01#		—0.75	8.9	— 8.9	25.0 ft.
	4*	2,000	8	—0.36	2.3	— 4.6	12.8	+0.01	—0.01	—0.36	2.3	— 4.6	2. Via Pipes
						—1.1 ÷ 40.1 × 1.85 = —0.01						— 0.3	3, 4, 5
II	4**	2,000	8	+0.36	2.3	+ 4.6	12.8	+0.01	—0.01	+0.36	2.6	+ 4.6	25.3 ft.
	5	1,000	6	+0.40	11.3	+11.3	28.2	+0.01		+0.41	11.8	+11.8	3. Via Pipes
	6	1,000	6	—0.40	11.3	—11.3	28.2	+0.01#		—0.39	10.8	—10.8	3, 6, 7
	7	2,000	6	—0.20	3.1	— 6.2	31.0	+0.01		—0.19	2.9	— 5.8	25.5 ft.
						—1.6 ÷ 100.2 × 1.85 = 0.01						— 0.2	

*Pipe serves more than one circuit; first consideration of this pipe.
**Second consideration of this pipe.
‡Corrections in this column are those calculated for the same pipe in the companion circuit; they are of opposite sign.
Q = Flow in m.g.d.; s = slope of hydraulic gradient or friction loss in ft. per 1,000 (‰) by the Williams and Hazen formula for $C = 100$.
H = Head lost in pipe (ft.).

q = Flow correction (m.g.d.) $q = -\frac{\text{Sum } H}{1.85 \text{ Sum } \frac{H}{Q}}$

$Q_c = Q_a + q$.

in the author's opinion, is the most generally useful one. As the analyst becomes familiar with the method, he will discover a number of different shortcuts, some of which have been described in the literature. He is warned, however, against the danger of employing shortcuts too early in the stepwise corrections, because that will inevitably lead to faulty results. As a matter of fact, shortcuts are ordinarily applicable only after successive corrections have balanced the network so closely that further correction, no matter how elegant and expeditious, lies well within the fundamental uncertainty introduced by the choice of such factors as the Williams and Hazen coefficient C.

To illustrate the application of the Hardy Cross method to a network of pipes, we may choose a simple system such as the one shown in Fig. 7. In spite of its simplicity, however, this network cannot be solved conveniently by algebraic methods, because it contains two interfering hydraulic constituents: (1) a cross-over (pipe No. 4) or pipe that operates in more than one circuit, and (2) a series of take-offs representing the flows used along the lines of pipe or actually drawn off through hydrants for fire fighting or through pipes into neighboring circuits. Turning to Table 4 and Fig. 7, the schedule of calculations includes the following:

Columns 1 to 4 identify the position of the pipes in the network and record their length and diameter. There are two circuits and seven pipes. Pipe No. 4 is shared by both circuits. To indicate this, it is starred in connection with Circuit I and double-starred in connection with Circuit II. The dual rôle of this pipe must not be overlooked.

Columns 5 to 9 deal with the assumed flows and the derived flow correction. For purposes of identification the hydraulic elements Q, s, H, and q are given a subscript zero.

Column 5 lists the assumed flows, Q_0, in m.g.d. They are preceded by positive signs if they are clockwise in direction and by negative signs if they are counterclockwise in direction. The distribution of flows has been purposely misjudged in order to highlight the balancing operation. At each junction the total flow remaining in the system must be accounted for.

Column 6 gives the friction losses, s_0, in ft. per 1,000 ft. when the pipe is carrying the quantities, Q_0, shown in column 5. The values of s_0 can be obtained directly from tables or diagrams of the Williams and Hazen formula. See Appendix.

Column 7 is obtained by multiplying the friction loss per 1,000 ft. of pipe (s_0) by the length of the pipe in 1,000 ft.; *i.e.* column 7 = column 6 × column 3 ÷ 1,000. The head losses, H_0, obtained are preceded by a positive sign if the flow is clockwise and by a negative sign if the flow is counterclockwise. The values in column 7 are added up for each circuit, with due regard to signs, in order to obtain "Sum H" in the flow-correction formula.

Column 8 is found by dividing column 7 by column 5. Division makes all signs of H/Q positive. This column is added up for each circuit in order to obtain "Sum H/Q" in the flow-correction formula.

Column 9 contains the calculated flow correction, q. The computations necessary to obtain

$$q = -(\text{Sum } H) \div (1.85 \times \text{Sum } H/Q)$$

are performed after the two summing operations that have been described in connection with columns 7 and 8. For example: in circuit I, Sum $H = -16.5$, Sum $H/Q = 43.1$; and $(-16.5) \div (1.85 \times 43.1) = -0.21$; or $q = +0.21$. Since pipe No. 4 operates in both circuits, it draws a correction from each circuit. The second correction, however, is of opposite sign to that applied to the companion circuit. As a part of circuit I, for example, pipe No. 4 receives a correction of $q = -0.07$ from circuit II in addition to its basic correction of $q = +0.21$ from circuit I.

Columns 10 to 14 cover the once-corrected flows. The hydraulic elements (Q, s, H, and q) are, therefore, given the subscript one. Column 10 is obtained by adding, with due regard to sign, columns 5 and 9. Columns 11, 12, 13, and 14 are then found in the same manner as columns 6, 7, 8, and 9.

Columns 15 to 19 record the twice corrected flows, and the hydraulic elements (Q, s, H, and q) carry the subscript two. These columns are otherwise like columns 10 to 14.

Columns 20 to 23 present the final result, columns 20 to 22 corresponding to columns 15 to 18 or 10 to 12. No further flow corrections are developed because the second flow corrections are of the order of 10,000 g.p.d. for a minimum flow of 200,000 g.p.d. or at most 5%. To test the balance obtained, the losses of head between points A and D in Fig. 7 via the three possible routes are given in column 23. The losses vary from 25.0 to 25.5 ft. The average loss is 25.3 ft., and the variation is about 1%.

In accordance with the principles set forth in the Appendix to this paper, doubling or halving the inflow (or changing it in any other ratio) will double or halve the flow in each pipe and at each take-off (or change it in corresponding ratio). The loss of head through the system, furthermore, will be altered as the 1.85 power (or approximately as the square) of the ratio. If the inflow is increased from 2.0 m.g.d. to 3.0 m.g.d., for example, the loss of head from A to D will become $25.3 \times 1.5^{1.85} = 54$ ft.* (approximately $25.3 \times 1.5 \times 1.5 = 57$ ft.). If the take-off flow is not changed in proportion to the inflow, the system must be balanced anew from the beginning.

It is not necessary to apply the Hardy Cross method to a large network as a whole. Much information can be obtained more quickly and more simply if the method is used to balance portions of the system in succession. Good judgment on the part of the analysts is an indispensable aid to network investigations such as these.

Method of Equivalent Pipes. By this method, a complex system of pipes is replaced by a single line of equivalent capacity. The method cannot be applied directly to a system of pipes that contains cross-overs or take-offs. It is frequently possible, however, by judicious skeletonizing of the system to obtain significant information on the quantity and pressure of water available at important points in the network. In paring down the system to a workable skeleton, the analyst can be guided by the fact that the following pipes contribute little to flow: (1) small pipes, 6 in. and under in most systems and as large as 8 or 10 in. in more extensive systems; and (2) pipes at right angles to the direction of flow for which no appreciable pressure differential is established between their junctions in the system. The method of equivalent pipes is useful in simplifying networks that are to be analyzed by the Hardy Cross method. Scattered through most networks there are often combinations of pipes that can be replaced by hydraulically equivalent pipes, thus reducing to advantage the number of units that must be handled in the Hardy Cross method.

The method of equivalent pipes makes use of two hydraulic axioms: (1) that the loss of head due to the flow of a given quantity of water through pipes in series, such as pipes AB and BD in Fig. 8, is additive,

*A simple way of finding the answer graphically from the Williams and Hazen Diagram is illustrated in Fig. 11f in the Appendix to this paper.

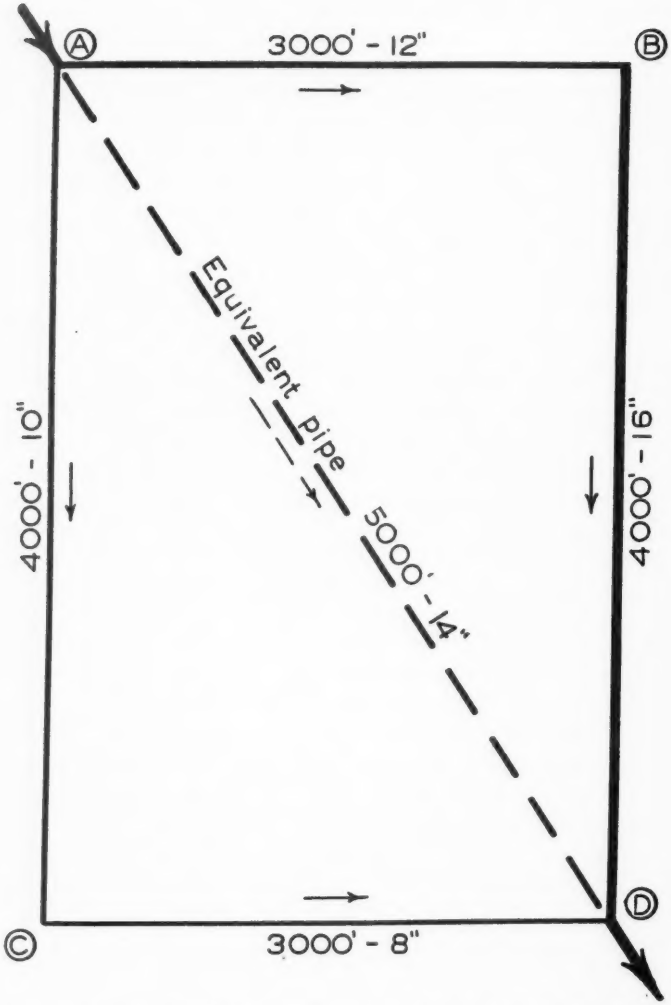


FIG. 8—PLAN OF NETWORK ANALYZED BY THE METHOD OF EQUIVALENT PIPES. See Table 5.

and (2) that the quantities of water flowing through pipes in parallel, such as pipes ABD and ACD in Fig. 8, must be such that the loss of head through each line is the same. Using the system shown in Fig. 8 as an illustration, the necessary calculations are summarized in Table 5.

TABLE 5—ANALYSIS OF A NETWORK (FIGURE 8) BY THE METHOD OF EQUIVALENT PIPES.

Q = flow in m.g.d.; s = friction in ft. per 1,000; H = loss of head in ft.; Williams and Hazen coefficient C = 100.

1. *Line ABD.* Assume Q = 1 m.g.d.
   a. Pipe AB, 3,000 ft., 12 in., s = 2.1, H = 2.1 × 3 = 6.3
   b. Pipe BD, 4,000 ft., 16 in., s = 0.52, H = 0.52 × 4 = 2.1
   c. Total H = 8.4
   d. Equivalent length of 12-in. pipe: 8.4 ÷ 2.1 = 4,000 ft.
2. *Line ACD.* Assume Q = 0.5 m.g.d.
   a. Pipe AC, 4,000 ft., 10 in., s = 1.42, H = 1.42 × 4 = 5.7
   b. Pipe CD, 3,000 ft., 8 in., s = 4.2, H = 4.2 × 3 = 12.6
   c. Total H = 18.3
   d. Equivalent length of 8-in. pipe: 18.3 ÷ 4.2 = 4,360 ft.
3. *Equivalent line AD.* Assume H = 8.4 ft.
   a. Line ABD, 4,000 ft., 12 in., s = 8.4 ÷ 4.00 = 2.1, Q = 1.00
   b. Line ACD, 4,360 ft., 8 in., s = 8.4 ÷ 4.36 = 1.92, Q = 0.33
   c. Total Q = 1.33
   d. Equivalent length of 14-in. pipe: Q = 1.33, s = 1.68, 8.4 ÷ 1.68 = 5,000 ft.
   e. Result: 5,000 ft. of 14-in. pipe.

The calculations involved in Table 5 may be outlined as follows:

1. Since line ABD consists of two pipes in series, the losses of head created by a given flow of water are additive. Find, therefore, from the Williams and Hazen diagram the frictional resistance, s, for some reasonable flow (1 m.g.d.), (a) in pipe AB and (b) in pipe BD. Multiply these resistances by the length of pipe to obtain the loss of head, H. Add the two losses to find the total loss H = 8.4 ft. Line ABD, therefore, must carry 1 m.g.d. with a total loss of head of 8.4 ft. Any pipe that will do this is an equivalent pipe. Since a 12-in. pipe has a resistance s = 2.1 ft. per 1,000 when it carries 1 m.g.d. of water, a 12-in. pipe, to be an equivalent pipe, must be 8.4 ÷ 2.1 = 4,000 ft. long.

2. Proceed for line ACD in the same general way as for line

ABD and find the length of the equivalent 8-in. pipe to be 4,360 ft.

3. Since ABD and ACD together constitute two lines in parallel, the flows through them that are associated with a given loss of head are additive. If some convenient loss is assumed, such as the loss already calculated for one of the lines, the missing, companion flow can be found from the Williams and Hazen diagram. Assuming a loss of 8.4 ft., which is associated with a flow through ABD of 1 m.g.d.,* it is only necessary to find from the diagram that the quantity of water that will flow through the equivalent pipe ACD, when the loss of head is 8.4 ft. or $s = 8.4 \div 4.36 = 1.92$ ft. per 1,000, amounts to 0.33 m.g.d. Add this quantity to the flow through line ABD (1.0 m.g.d.) and obtain 1.33 m.g.d. Line AD, therefore, must carry 1.33 m.g.d. with a loss of head of 8.4 ft. If the equivalent pipe is assumed to be 14 in. in diameter, it will discharge 1.33 m.g.d. with a frictional resistance $s = 1.68$ ft. per 1,000, and its length must be $8.4 \div 1.68 = 5{,}000$ ft. We can therefore replace the network shown in Fig. 8 by a single 14-in. pipe 5,000 ft. long.

No matter what the original assumptions for quantity, diameter and loss of head may be, the calculated equivalent pipe will always perform hydraulically in exactly the same way as the network that it replaces.

Other Methods of Analysis. There are a number of other methods for studying the hydraulics of networks in the office. Among them should be mentioned in particular Freeman's graphical method as expanded by Howland* and Camp and Hazen's† hydraulic analysis of distribution systems by an electric analyzer. In the author's opinion, neither one of these methods is as satisfying as the Hardy Cross method.

Service Storage

Service, or distribution, reservoirs are hydraulically an integral part of the distribution system. The supply conduits leading to them are generally so proportioned that they can deliver water at a rate that is sufficiently high to meet the 24-hour demand of the maximum day. Hourly demands in excess of this rate are supplied from storage to which must be added the water reserves needed in case of

*It was, therefore, really unnecessary to specify the length and diameter of the equivalent pipe ABD.
*This Journal, **48** (1934), 408.
†This Journal, **48** (1934), 383.

a serious conflagration. Depending upon the nature of the supply works and particularly upon the character of the supply conduits—number, length, and material, and the presence of special hazards such as river crossings and unstable ground—further volumes of water must be stored to allow for the interruption of supply for repairs to conduits and other works. The three major components of service storage, therefore, are: (1) equalizing or operating storage; (2) fire reserve; and (3) emergency reserve. In what amounts they must be provided will be outlined in the following.

Equalizing or Operating Storage. If the planned rate of supply and the fluctuation in the rate of demand are known, the equalizing or operating storage that should be provided may be ascertained from a rate curve or, more satisfactorily, from a mass diagram.* As shown in Fig. 9 for the simple conditions of steady inflow, during 12 and 24 hours respectively, the amount of operating or equalizing storage is the sum of the maximum ordinates between the demand and supply lines. To construct a mass diagram proceed as follows:

1. From past measurements of flow, determine the draft during each hour of the day and night for typical days (maximum, average and minimum). Let us say that these drafts, as in Fig. 9, are as follows for 4-hourly periods beginning at midnight:

Time	Midnight-4 a.m.	4-8 a.m.	8 a.m.-Noon	Noon-4 p.m.	4-8 p.m.	8 p.m.-Midnight
Draft—m.g.	0.484	0.874	1.216	1.102	0.818	0.506

2. Calculate the amounts of water that are drawn up to certain times, *i.e.*, the cumulative draft.

Time	4 a.m.	8 a.m.	Noon	4 p.m.	8 p.m.	Midnight
Cumulative Draft—m.g.	0.484	1.358	2.574	3.676	4.494	5.000

3. Plot the cumulative draft against time, as shown in Fig. 9.

4. For steady supply during 24 hours, draw a straight line diagonally across the diagram as in Fig. 9a. The ordinates between the draft and the supply line measure the difference between demand and supply. By 6 a.m., the supply exceeds the demand by 0.40 m.g.,

*Similar to the Rippl diagram employed in the study of storage to be provided in impounding reservoirs.

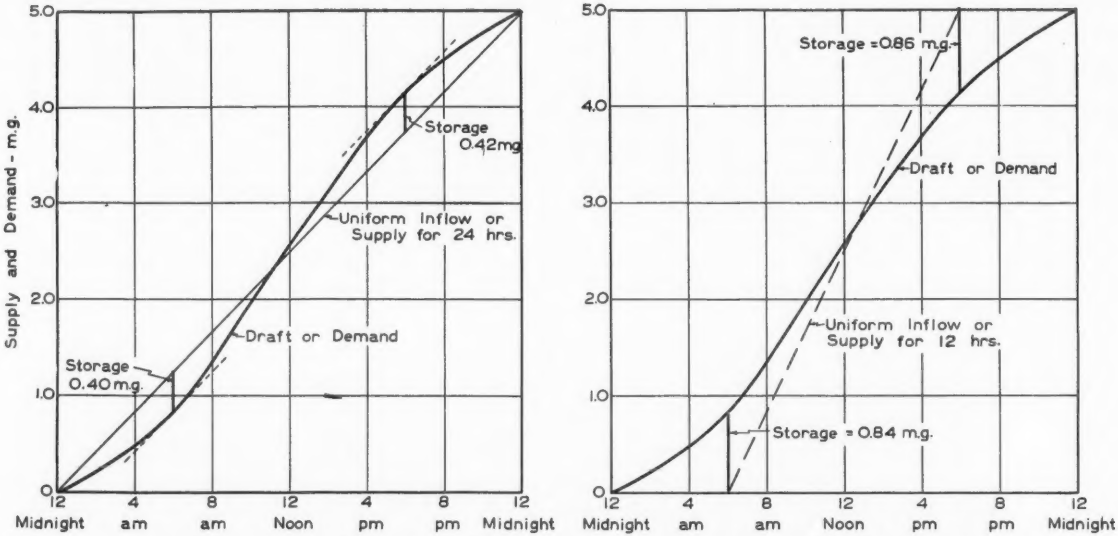


FIG. 9—DETERMINATION OF EQUALIZING, OR OPERATING, STORAGE BY MASS DIAGRAM.
a. Uniform inflow, or supply, extending over 24 hours.
b. Uniform inflow, or supply, confined to 12 hours.

but by 11 a.m. this excess has been used up, assuming that it was stored. This, therefore, is the first component of the required storage. By 6 p.m., the demand exceeds the supply by 0.42 m.g. This amount, therefore, is the second component of required storage. The total storage required is the sum of the two components, or 0.82 m.g.

5. For steady supply during 12 hours, by pumping, for example, draw a straight line diagonally from the beginning of the pumping period to its end (from 6 a.m. to 6 p.m. in Fig. 9b). The draft of 0.84 m.g. exerted between midnight and 6 a.m. must be supplied from storage as must the draft of 0.86 m.g. from 6 p.m. to midnight. Total storage, therefore, is 1.70 m.g.

Steady supply at the rate of maximum daily use will ordinarily require an equalizing storage close to 15% of the average day's consumption. Limitation of supply to 12 hours may be expected to raise the operating storage to 50% of the average day's consumption. The values shown in Fig. 9 are 17% and 34% respectively.

Fire Reserve. Based upon the duration of serious conflagrations that have been experienced in the past, the National Board of Fire Underwriters recommends that distributing reservoirs be made large enough to supply water for fighting a serious conflagration for ten hours in communities of more than 2,500 people and for five hours in smaller ones. The resulting fire reserve is shown in Table 1. These amounts may not always be economically attainable, and design values may have to be adjusted downward to meet local financial abilities.

Emergency Reserve. The magnitude of this component of storage depends upon the danger of interruption of reservoir inflow by failure of supply works, and time needed to make repairs. If the shutdown of the supply is confined to the time necessary for routine inspections and these are relegated to the hours of minimum draft, the emergency reserve is sometimes taken as equal to 25% of the total storage capacity, *i.e.,* the reservoir is assumed to be drawn down by one-quarter of its average depth. If supply lines or equipment are expected to be out of operation for longer periods of time, suitable allowances must be made. The National Board of Fire Underwriters bases its rating system on an emergency storage equal to five days of maximum flow.

Total Storage. The total amount of storage is desirably equal

to the sum of the component requirements. In each instance, economic considerations determine the final choice. In pumped supplies, cost of storage must be balanced against cost of pumping with particular attention to the economies that can be effected by more uniform operation of pumps and restricting pumping to a portion of the day. In all supplies, cost of storage must be balanced against cost of supply lines, increased fire protection and more uniform pressures in the distribution system.

For a steady gravity supply equal to the maximum daily demand, a ten-hour fire supply, and no particular hazard to the supply works, the storage to be provided for a city of 50,000 people using an average of 5 m.g.d. of water would approximate the following amounts:

Equalizing storage = 15% of 5 m.g.	0.75 m.g.*
Fire reserve (Table 1)	4.00 m.g.
Sub-total	4.75 m.g.
Emergency reserve = ¼ of total storage. Therefore sub-total is ¾ of total storage, and	
Total storage = 4.75 ÷ 0.75	6.33 m.g.

If we assume that the maximum daily use is 140% of the average, the emergency storage suggested by the National Board would approximate $5 \times 5 \times 1.4 = 35$ m.g. instead of $6.33 - 4.75 = 1.58$ m.g.

Location of Storage. As shown earlier in this paper (Fig. 3), location as well as capacity of service storage is an important factor in the control of distribution systems. A million gallons of elevated fire reserve, suitably situated with reference to the area that is to be protected, is equivalent, for example, to the addition of a 12-in. supply main. If this volume of water is drawn in a 10-hour fire, flow would be provided at a rate of $(24 \div 10) \times 1 = 2.4$ m.g.d. This is the amount of water that a 12-in. pipe can carry at a velocity under 5 ft. per sec. That this must be neighborhood storage, however, is evidenced by the high frictional resistance of more than 10 ft. per 1,000 that accompanies such use.

Conclusion. The importance of the water-supply mains and laterals in the network of streets of our communities has been made

*The illustrative example gave 17% or 0.82 m.g.

manifest, more than ever before, by the part that water supply is called upon to play in time of war. The hydraulic investigation of existing systems and their reinforcement, therefore, is today not only a question of public service but also a matter of national defense.

This paper has been prepared with the hope that it may contribute in some measure to the strengthening of American water works for national defense by aiding in the re-examination of water-supply networks in the field and in the office.

In closing, the author wishes to express his sincere appreciation to Richard H. Ellis, Inspection Department, Associated Factory Mutual Fire Insurance Companies, for reading the draft of this paper and offering valuable suggestions for its correction and amplification.

APPENDIX

The Hardy Cross Method of Balancing Networks

Derivation of Basic Equation. In the simple network of pipes of known length, size, and condition shown in Fig. 10a, the incoming

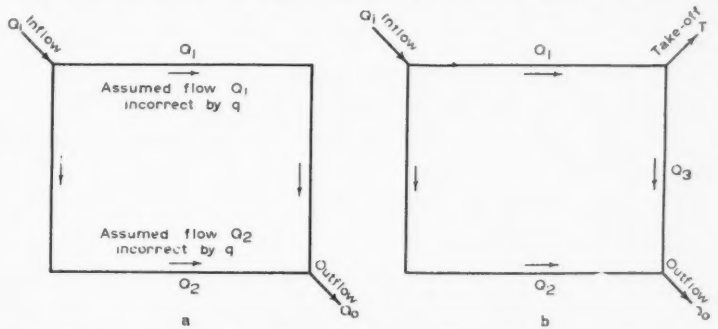


Fig. 10—Simple Networks Illustrating (a) the Derivation of the Hardy Cross Method and (b) the Effect of Changing Flows.

flow of water, Q, is assumed to be split between the two branches so that the clockwise flow is Q_1 and the counterclockwise flow is $Q_2 = Q - Q_1$. Calculation of the loss of head in the two branches by some convenient pipe-flow formula yields a loss of head due to the clockwise flow, Q_1, of H_1 and a loss of head due to the counterclockwise flow, Q_2, of H_2. In accordance with any one of the commonly used

exponential formulas for the flow of water in pipes, the loss of head is $H = k\,Q^n$,* where k is a numerical constant for a particular pipe and n is a constant for all pipes. If in Fig. 10a, Q_1 and Q_2 are chosen correctly, $H_1 = k_1\,Q_1^n$ must equal $H_2 = k_2\,Q_2^n$, or $H_1 - H_2 = 0$. Here losses associated with clockwise flow are arbitrarily given a positive sign and losses associated with counterclockwise flow a negative sign. If, as may be expected, $H_1 - H_2$ is not zero, the initially assumed values of Q_1 and Q_2 must be in error. If Q_1 is too small by an amount q, then Q_2 must be too large by the same amount q, because the flows in the two branches must always balance, *i.e.*, add up to a total of Q. If the corrected value of Q_1 is called $Q_1' = Q_1 + q$ and the corrected value of Q_2 is called $Q_2' = Q_2 - q$, the losses of head, H_1' and H_2', can be calculated. Since the frictional resistances through both branches must be the same, the difference between these losses must equal zero, or:

$$H_1' - H_2' = k_1\,(Q_1 + q)^n - k_2\,(Q_2 - q)^n = 0 \qquad (1)$$

Binomial expansion of $(Q_1 + q)^n$ and $(Q_2 - q)^n$ transforms equation (1) into the following form:

$$k_1(Q_1^n + n\,q\,Q_1^{n-1} + \ldots) - k^2(Q_2^n - n\,q\,Q_2^{n-1} + \ldots) = 0$$

If the first estimate of flow distribution has been reasonable, q will be small and the terms in the expansion that include q^2, q^3, etc. (not shown above) will be so small as to be safely neglected. We may write, therefore:

$$k_1\,Q_1^n + n\,k_1\,q\,Q_1^{n-1} - k_2\,Q_2^n - n\,k_2\,q\,Q_2^{n-1} = 0,$$

and solving for q, we find:

$$q = -\frac{k_1Q_1^n - k_2Q_2^n}{nk_1Q_1^{n-1} + nk_2Q_2^{n-1}}. \qquad (2)$$

But $k_1Q_1^n = H_1$ and $k_2Q_2^n = H_2$; also $nk_1Q_1^{n-1} = \dfrac{nk_1Q_1^n}{Q_1} = \dfrac{H_1}{Q_1}$ and $nk_2Q_2^{n-1} = \dfrac{nk_2Q_2^n}{Q_2} = \dfrac{H_2}{Q_2}$ where Q_1 and Q_2 are given positive or negative signs, in the same manner as H_1 and H_2, for clockwise and counter-

*In the Williams and Hazen formula $Q = 405\ C\ d^{2.63}\ s^{0.54}$, or for given values of C and d, and for $s = H/C$, $H = k\ Q^{1.85}$.

clockwise flow. Substitution of these expressions in equation (2) gives

$$q = -\frac{H_1 - H_2}{n\left(\frac{H_1}{Q_1} + \frac{H_2}{Q_2}\right)} \tag{3}$$

Since $H_1 - H_2$ represents the sum of the losses of head and $\frac{H_1}{Q_1} + \frac{H_2}{Q_2}$ the sum of the three quotients, and since $n = 1.85$ in the Williams-Hazen formula, we may write the general equation for the flow correction, as used in the body of this paper, as follows:

$$q = -\frac{\text{Sum } H}{1.85 \text{ Sum } \frac{H}{Q}} \tag{4}$$

Effect of Changing Flows. If the total flow through the network is changed, the distribution of flows between individual pipes will be changed in the same proportion, provided that the take-off flows are also changed in the same ratio. The losses of head that accompany such changes vary in proportion to the 1.85 power (or approximately as the square) of the ratio of change. That this is so can be shown as follows: In Fig. 10b, $Q_i = Q_1 + Q_2$. If Q_i is changed to $K\,Q_i$ where K is the ratio of the new to the old flow,

$$K\,Q_i = K\,Q_1 + K\,Q_2;$$
$$\text{also } K\,Q_0 = K\,Q_3 + K\,Q_2 = K(Q_1 - T) + K\,Q_2$$
$$\text{or } K\,Q_0 = K\,Q_1 + K\,Q_2 - K\,T \tag{5}$$

Furthermore, $H_1 = k_1\,Q_1^n$ as shown before, and

$$H_1' = k_1(K\,Q_1)^n, \text{ or}$$
$$\frac{H_1'}{H_1} = K^n = K^{1.85} \tag{6}$$

Williams and Hazen Diagram for the Solution of Pipe Flow Problems

In order that this paper may be self-sufficient—so far as this is possible in connection with so broad a subject as the hydraulics of networks—a reproduction of a diagram based upon the Williams

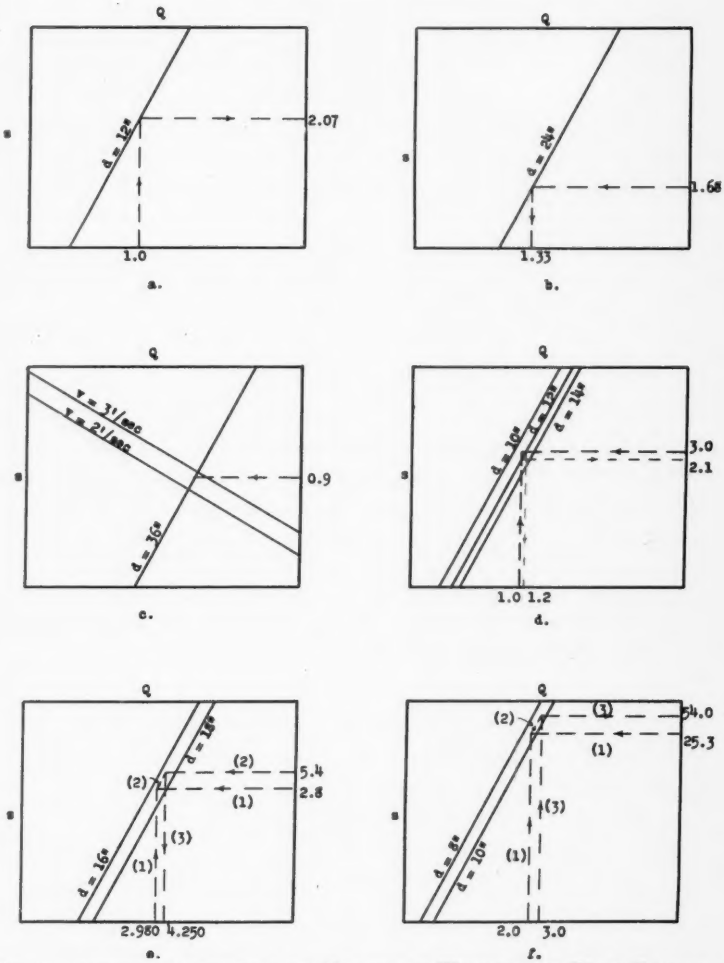
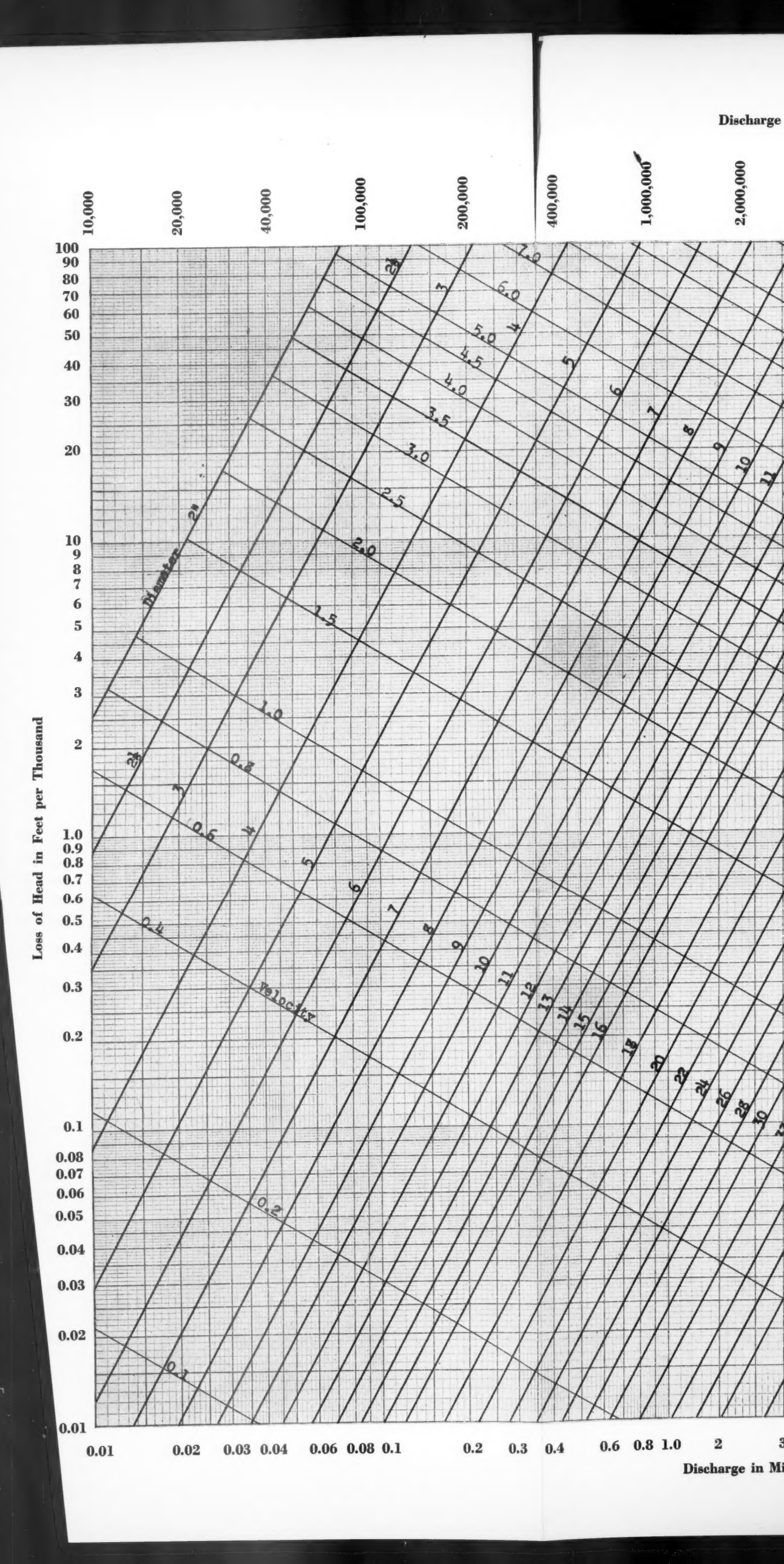


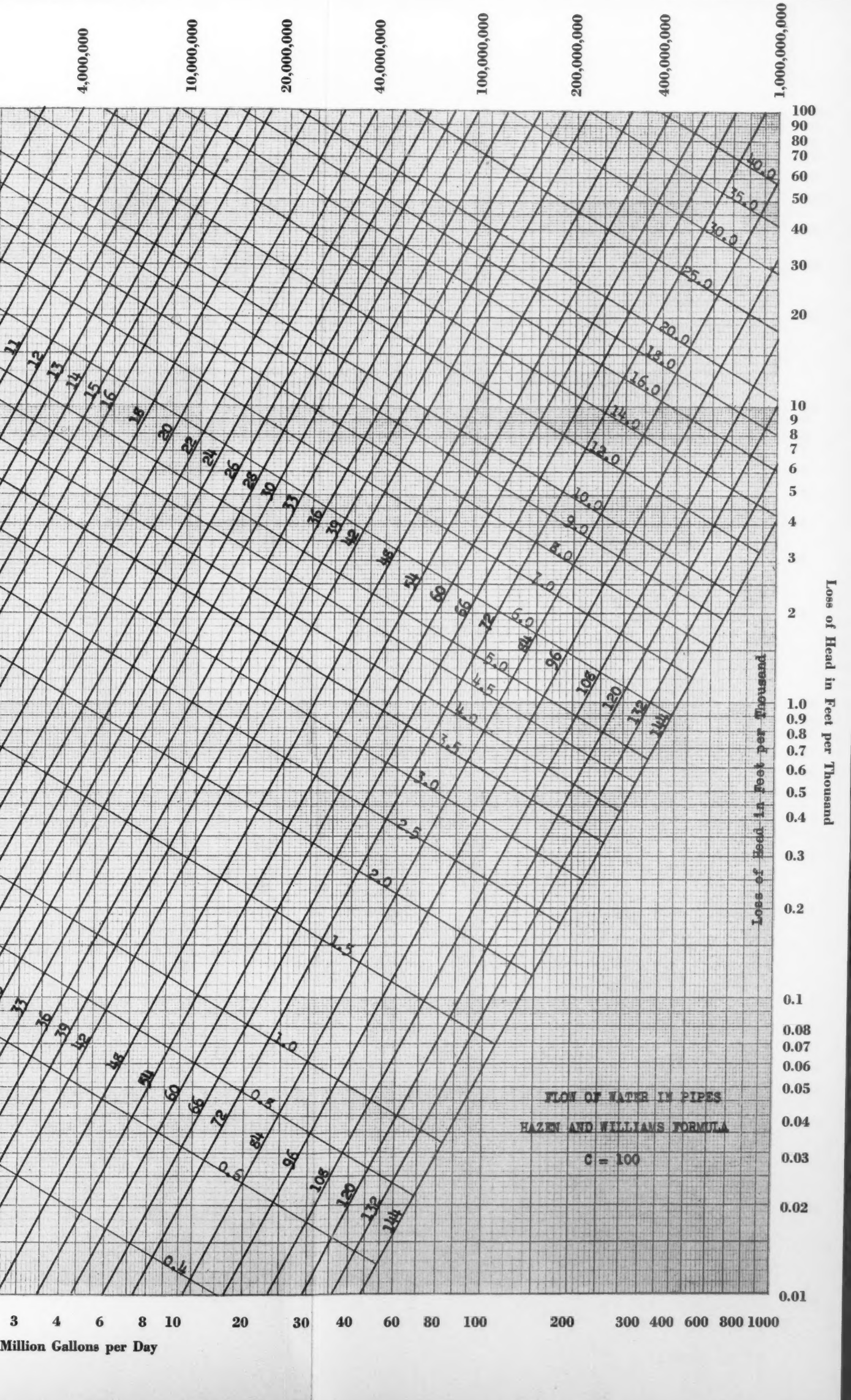
FIG. 11—SKETCHES ILLUSTRATING THE USE OF THE WILLIAMS AND HAZEN DIAGRAM FOR THE FLOW OF WATER THROUGH PIPES.

a. Given Q, d, and l; to find s and H.
b. Given D and s, or H and l; to find Q.
c. Given D and s, or H and l; to find v.
d. Given Q and s, or H and l; to find d.
e. Given Q and H; to find Q for a different H.
f. Given Q and H; to find H for a different Q.


Discharge
10,000
20,000
40,000
100,000
200,000
400,000
1,000,000
2,000,000
Loss of Head in Feet per Thousand
Diameter
Velocity
Discharge in Mi

harge in Gallons per Day

Fair on Distribution Systems



Loss of Head in Feet per Thousand

and Hazen formula is included, drawn to a sufficiently large scale to make it readable. Some of the normal uses of a diagram such as this are outlined below and illustrated in the thumbnail sketches of Fig. 11 which are intended to serve as guides to the necessary procedures. As drawn, the diagram is confined to conditions of flow for which the discharge coefficient C in the Williams-Hazen formula is 100. So as not to confuse the main issue, a value of $C = 100$ has been assumed to apply in all calculations that have been developed in this paper. How to adjust results to different magnitudes of C is discussed briefly at the end of the six primary examples.

a. Given a 12-in. pipe 2,000 ft. long discharging 1.0 m.g.d.; to find the loss of head.

As in Fig. 11a find, for $Q = 1.0$ and $d = 12$, $s = 2.1$ ft. per 1,000, and calculate $H = s \times C = 2.1 \times 2 = 4.2$ ft.

This is the calculation for pipe 1 in circuit I of Table 4.

b. Given a 24-in. pipe 5,000 ft. long with a loss of head of 7.0 ft.; to find the discharge.

Calculate $s = H \div C = 7 \div 5 = 1.4$ ft. per 1,000 and find as in Fig. 11b, for $s = 1.4$ and $d = 24$, $Q = 5.0$ m.g.d.

c. Given a loss of head of 5.4 ft. for a 36-in. pipe 6,000 ft. long; to find the velocity.

As in Fig. 11c, follow the horizontal through $s = 5.4 \div 6 = 0.9$ to its intersection with the diameter line $d = 36$ and estimate by interpolation between the velocity lines $v = 2.5$ ft. per sec.

d. Given a discharge of 1.0 m.g.d. and a loss of head of 9 ft. in a distance of 3,000 ft.; to find the commercial size of a pipe that will carry this amount of water with a loss of head equal to or less than the available amount.

Calculate $s = H \div C = 9 \div 3 = 3$ ft. per 1,000 and find as in Fig. 11d, for $s = 3.0$ and $Q = 1.0$, $d = 12$ in.

As indicated in Fig. 11d, this pipe can either discharge $Q = 1.0$ m.g.d. with a loss of head of $2.1 \times 3 = 6.3$ ft., or $Q = 1.2$ m.g.d. with a loss of head $H = 3.0 \times 3 = 9.0$ ft.

e. Given a discharge of 2.980 g.p.m. for a pressure drop of 28 lb. per sq. in.; to find the discharge for a pressure drop of 54 lb. per sq. in.

As in Fig. 11e, (1) find the intersection of the vertical through any Q value of 2,980 (29.8, 2.98, or 0.298) with the horizontal

through any s value of 28 (28, 2.8, or 0.28); (2) from this point follow parallel to the inclined diameter lines to an intersection with the horizontal through any s value of 54 (54, 5.4, or 0.54 respectively); (3) from this intersection drop vertically to the desired Q value of 4,250 g.p.m. This is the calculation necessary to determine the delivery of engine streams by the group of hydrants of Fig. 5 and Table 2.

f. Given a flow of 2.0 m.g.d. entering a system of pipes and a loss of head of 25.3 ft. between two points in the system. To find the loss of head when the flow is increased to 3.0 m.g.d.

As in Fig. 11f, (1) find the intersection of the vertical through $Q = 2.0$ m.g.d. with the horizontal through $s = 25.3$; (2) from this point follow parallel to the inclined diameter lines to an intersection with the vertical through $Q = 3.0$ m.g.d.; (3) from this intersection proceed horizontally to the desired value of $s = 54$. This is the calculation performed in connection with changing the inflow to the system (Fig. 7) analyzed by the Hardy Cross method.

In the absence of specific information on the capacity coefficient C in the Williams and Hazen formula, a value of $C = 100$ is commonly assumed. For values other than 100, the following additional steps must be taken in the use of the diagram:

1. When the discharge, Q, is given, divide the value of Q, before entering the diagram, by one-hundredth of the actual C. If, for example, the discharge is 2 m.g.d. and $C = 120$, use in the diagram, in place of $Q = 2$ m.g.d., a value $Q = 2 \div 1.2 = 1.67$ m.g.d. All other procedures remain unchanged.

2. When the discharge, Q, is to be found, determine from the diagram the discharge that would obtain for $C = 100$ and multiply this value by one-hundredth of the actual C. If, for example, the discharge read from the diagram is 3 m.g.d. and the pipe coefficient is $C = 90$, calculate Q as $3 \times 0.9 = 2.7$ m.g.d.

PROCEEDINGS

FEBRUARY MEETING

HOTEL STATLER, BOSTON, MASS.

Thursday, February 20, 1941.

President Leland G. Carlton in the chair.

Secretary GIFFORD. At the Executive Committee meeting this morning the following were elected to the Association.

Members: Edward B. Bell, Chemist, Lowell, Mass.; Laban Clough Morrill, Town Engineer, Stoughton, Mass.; Robert W. Sayles, Jr., Treasurer, Baltic Water Co., Baltic, Conn.

Associates: Calgon, Inc., Pittsburgh, Pa.; American Smelting & Refining Co., Cambridge, Mass.

A paper "Coöperation of Fire and Water Departments in Planning for Emergencies," was read by Samuel J. Pope, Chief, Boston Fire Department.

HENRY T. GIDLEY presented the following motion to be sent to Edward R. Stettinius, Jr., Director, Priorities Division, Officer of the Production Manager, Washington, D. C.

WHEREAS, the maintenance of a public water supply is essential to any community, the members of the New England Water Works Association are of the opinion that in case of emergency, as in accidents to pumping stations, reservoirs, filter beds, standpipes, and breaks in pipe lines, orders for repair materials should be given equal priority with defense materials, such orders, to be valid, to be attested by authorized officials connected with the water works department, either public or private, in such communities so ordering.

[The question was put and the resolution was adopted.]

A paper "Recent Pollution of the Rochester Public Water Supply through Cross Connections," was read by Earl Devendorf, Assistant Director, Division of Sanitation, New York State Department of Health. Richard H. Ellis submitted a written discussion, and the paper was also discussed by Messrs. E. Sherman Chase, Warren J. Scott, and Gordon M. Fair.

A paper "Pumping Salt Water into a Water Distribution System," was read by Francis H. Kingsbury, Senior Assistant Sanitary Engineer, Massachusetts Department of Public Health. A question was asked by Allen M. Symonds.

MARCH MEETING

HOTEL STATLER, BOSTON, MASS.

Thursday, March 20, 1941.

President Leland G. Carlton in the chair.

Secretary GIFFORD. At the Executive Committee meeting this morning the following were elected to membership in the Association: Ernest E. Erickson, Water Commissioner, Milton, Mass.; and Kenneth H. Holmes, City Engineer, New London, Conn. You will recall that at the last meeting we passed a resolution on Priorities. Under date of February 28, I have received the following communication

"We thank you very much for sending me a copy of the resolution adopted at a meeting of the New England Water Works Association on February 20. We are glad to have this information and wish to assure you that any specific requests arising from unforeseen circumstances will be given our careful attention.

Sincerely yours,

E. R. STETTINIUS, JR.,
Director of Priorities."

Subsequently we received a communication, through Mr. Enslow, from the Office of Production Manager, which reads as follows:

"I wish to thank you for the stand you have taken in regard to the question of Priority for material and equipment required for public water utilities.

"As indicated in Mr. Stettinius's letter to the Honorable Harry Irvine, Mayor of Cumberland, Maryland, it is our general policy to limit the issuance of priorities to specific cases where defense material is involved. There is no question of not providing for the issuance of priorities to public water supply utilities—it is simply a matter of asking the water companies to comply with the general procedure which has been established for those requiring priority issues.

"You may be assured that if any water works find difficulty in procuring either material or equipment, their application for priority rating on the specific material or equipment will receive the prompt attention of this division. Application should be made directly to the Priorities Division on forms which are provided for that purpose, and which are available upon request. In the case of emergency purchases, arrangement can often be made by telegram or telephone, but this procedure should be avoided whenever possible.

Sincerely yours,

BLACKWELL SMITH,
Assistant Director, Priorities Division."

The PRESIDENT. At the meeting of the Executive Committee last month the following amendment to the constitution was submitted:

"At a meeting of the Executive Committee to be held in October of each year, the Executive Committee of the Association shall appoint the members of all standing professional and technical committees of the Association or shall delegate the duty of appointing any or all of such committees to the President, except as otherwise provided in the Constitution. A list of all such committee members shall be forwarded to each committee member by the Secretary in November. In case such appointments are made by the President, he shall notify the Secretary of his appointments on or before the first week in November. During the month of November the Secretary shall send a list of committee members to the Editor of the JOURNAL, who shall arrange for its publication in the December issue of the JOURNAL. Interim committee appointments shall be made by the Executive Committee or as otherwise directed by vote of the Association and notification of such committee members shall be made by the Secretary within fifteen days after their appointment."

On motion of Percy A. Shaw, duly seconded, it was Voted to amend Article VI by adding after Section 3 a new Section 3a as above.

On motion of Harold L. Brigham, duly seconded, it was Voted that the selection of the Nominating Committee be left with the President.

ROGER W. ESTY read the following letter, received by him from the American Water Works Association:

"Please give this letter your careful and prompt attention. It is addressed to you personally for the reason that I am soliciting your personal co-operation in a vital matter.

"Following a series of changes in the administrative control of the water supply service in the Army training areas, we are now advised that this work is assigned to Lt. Col. G. F. Lewis of the Utilities Maintenance and Repair Branch of the Quartermaster Corps.

"I have the following recent communication from him:

" 'Any help from your organization in bringing to our attention the names of qualified utility technicians who have either a Reserve Officer or a Civil Service status would be very much appreciated. . . . The administration of the work will be in the hands of officers selected for municipal utility experience. An operations and maintenance organization will be established at each camp. A technical staff and all key positions will be filled with qualified Civil Service civilians and if pending appropriation bills are passed by Congress the secondary and working staff of these groups will also be civilians.'

"We are not interested in the subject of civilian personnel at the present time—that may come later.

"It is highly important at this time for you to file with this office the names of any water works men now in military service. Particular interest is

indicated in men of officer status. In all cases please be particular to give the man's complete Army address, that is, his rank, company, camp, etc., and also his former civilian occupation."

ARTHUR C. KING commented on bills pending before the legislatures of Massachusetts and Connecticut.

PERCY R. SANDERS presented a resolution that the President be authorized to appoint a Committee to Study the Financing of New Developments and Extensions.

The motion was duly seconded and carried.

A paper "The Sanitary Significance of Coliform Bacteria in Water," was read by Dr. C. A. Stuart, Associate Professor of Biology, Brown University, Providence, R. I.

A paper "Comparative Efficiencies of Common Methods of Filter Washing," was read by Joseph Wuraftic, Engineer in Charge, Industrial Sanitation Section, Rhode Island Department of Health, Providence, Rhode Island.

A paper "Hydraulic Investigation of Water Distribution Systems in Field and Office," was read by Professor Gordon M. Fair, Harvard Graduate School of Engineering, Cambridge, Mass.

[Adjourned.]

Joint Meeting with the New England Health Institute and the New England Sewage Works Association
Hotel Statler, Boston, Mass.

Thursday, April 3, 1941.

Mr. Arthur D. Weston was General Chairman of this meeting. President Leland G. Carlton presided at the morning session and Walter J. Shea, President of the New England Sewage Works Association, at the afternoon session.

The following papers were presented:

"Military Field Sanitation" by Colonel William A. Hardenbergh, Sanitary Corps, U. S. Army, Washington, D. C.

"Bacteriologic Contaminants of Drinking Water" by W. LeRoy Mallman, Michigan State College, East Lansing, Mich.

"Toxic Contaminants of Drinking Water" by Lawrence T. Fairhall, National Institute of Health, U. S. Public Health Service, Washington, D. C.

"Symposium on Sanitary Personnel for National Defense" by Colonel John J. Reddy, Medical Corps, U. S. Army; Captain Joseph J. A. McMullen, Medical Corps, U. S. Navy; and Ralph E. Tarbett, U. S. Public Health Service.

"Development of Water Supply and Sewerage for 30,000 Troops at Camp Edwards" by Samuel M. Ellsworth, Consulting Engineer, Boston, Mass.

"Emergency Protection of Public Works and Utilities" by William W. Brush, Editor, *Water Works Engineering*, New York, N. Y.

"Sanitary Problems in the Vicinity of Troop Areas" by Walter D. Tiedeman, New York State Department of Health, Albany, N. Y.

On Friday, April 4, members of the Association took part in a trip to Fort Devens to examine the sanitary works of this camp. Mr. Frank A. Barbour, in charge of construction of Fort Devens, gave a brief description of the works. Some members continued the trip to Leominster, Mass., and visited the water filtration plant of that city.

OBITUARY

George Hall Hazlehurst

George Hall Hazlehurst, Chief Engineer and Director of the Bureau of Sanitation of the Alabama State Department of Public Health, died on March 7, 1941, at his home in Montgomery.

Mr. Hazlehurst was born on September 27, 1887, at Macon, Georgia. His early education was received in Barton Academy in Mobile, Alabama. After having studied at the Georgia School of Technology for a short time he returned to Rensselaer Polytechnic Institute where he had begun his college training and was graduated in 1910 with the degree of Civil Engineer. He later attended the Graduate School of Applied Science at Harvard University where in 1913 he was awarded the degree of Master of Civil Engineering.

Mr. Hazlehurst was one of the outstanding sanitary engineers in the South and was recognized as a leader in the field of sanitation not only in Alabama but throughout the country. He became affiliated with the State of Alabama on November 1, 1917, shortly after Dr. Samuel W. Welch became State Health Officer. Prior to this time Mr. Hazlehurst had been resident engineer on several large water plants, water distribution systems, and sewer systems in this country. He had been on the staff of the American Water Softener Company in Philadelphia, Sanitary Engineer for the Atlantic Coast Line Railway, and associated with his father J. N. Hazlehurst in Atlanta, Georgia. During the early part of the first World War he served as Sanitary Engineer with the Rockefeller Foundation in Servia. His diversified training and experience equipped him well for the position of chief sanitary engineer for the State of Alabama.

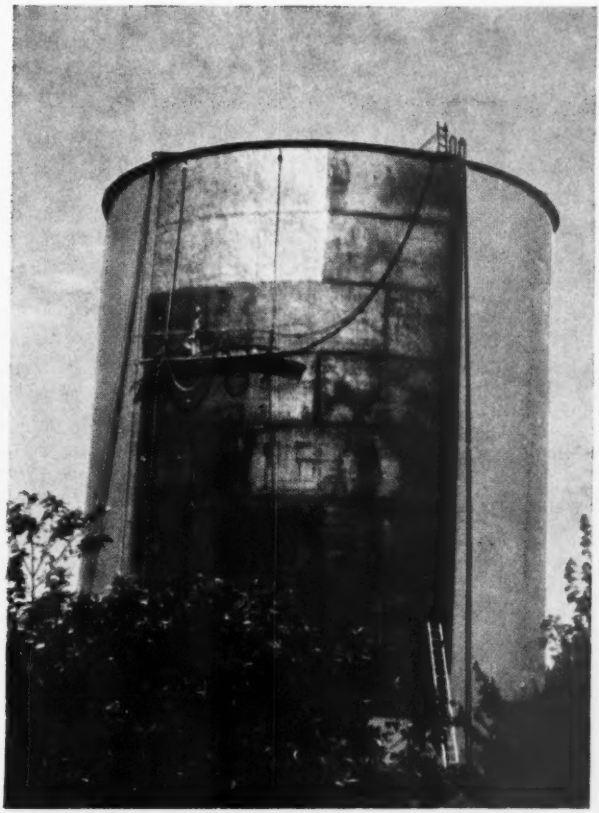
During twenty-three years of faithful service to the cause of public health Mr. Hazlehurst was one of the leaders in the building of a public health organization of the first order for the State of Alabama. In his work he not only met the exigencies of the moment, but with vision and clear thinking he built for the future. It was largely due to his efforts that Alabama cities and towns were brought to

realize their responsibility for providing adequate sanitation for all their inhabitants, regardless of their economic status. He was also instrumental in the setting up of legal machinery to enable them to assume this responsibility. In the field of malaria control he hardly had a peer in the whole South. Much of the credit for improved health conditions among the people of Alabama, especially with reference to the environmental diseases, is due him for his pioneering work in the field of environmental sanitation.

Mr. Hazlehurst joined the New England Water Works Association in 1913. He was also a member of the American Water Works Association, the American Public Health Association, the National Malaria Committee, the Conference of State Sanitary Engineers, and the Board of Malaria Consultants of the Tennessee Valley Authority.

The engineering profession has lost a competent engineer; the State of Alabama an outstanding worker for the betterment of its people; and those who worked with him a leader, counselor, and irreplaceable friend.

ARTHUR N. BECK
GORDON M. FAIR



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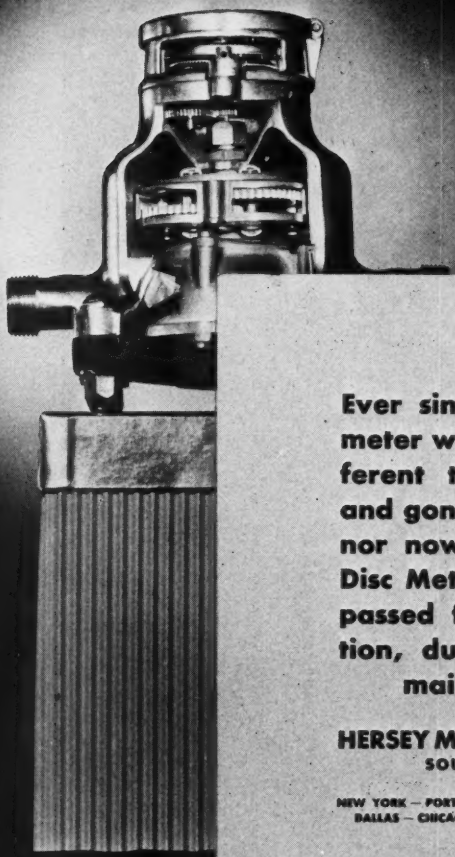

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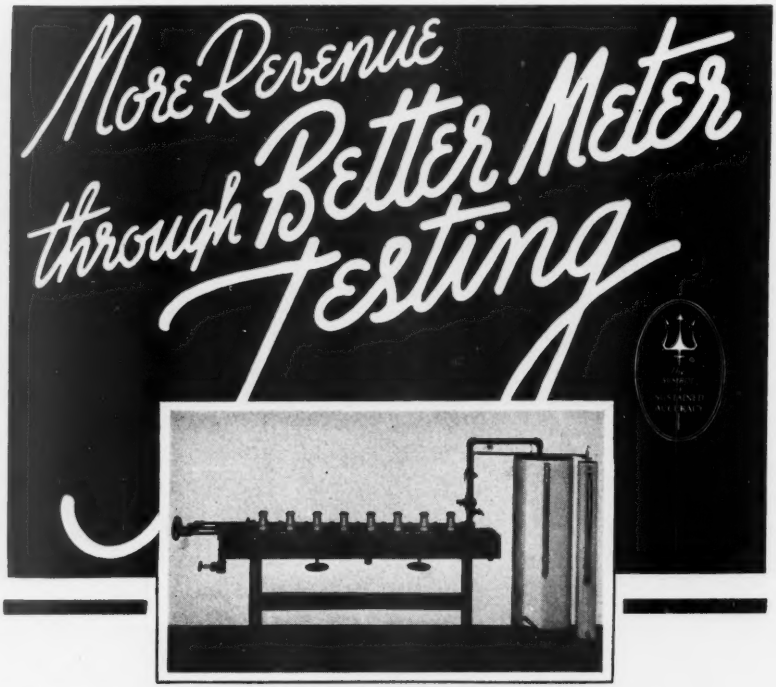

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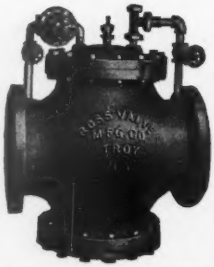

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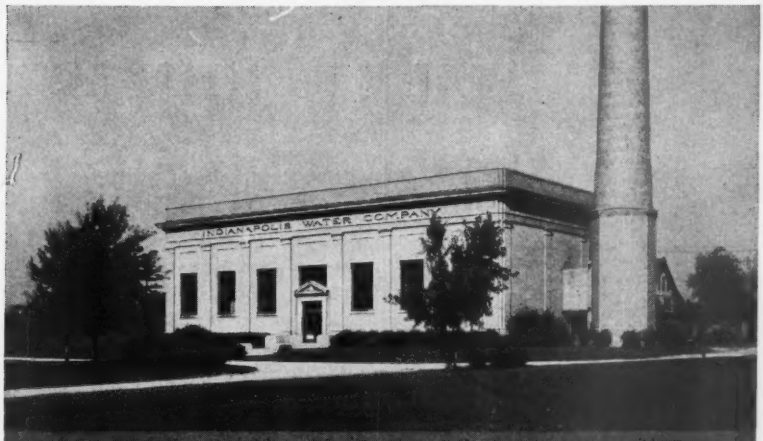

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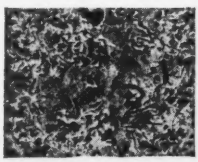
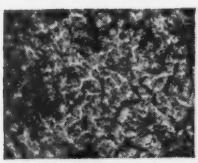


Photomicrograph (mag. 100x) showing the structure of a typical sample of cast iron as ordinarily produced. Note the entrained inclusions tending to break down the strength of the metal and produce leaks.



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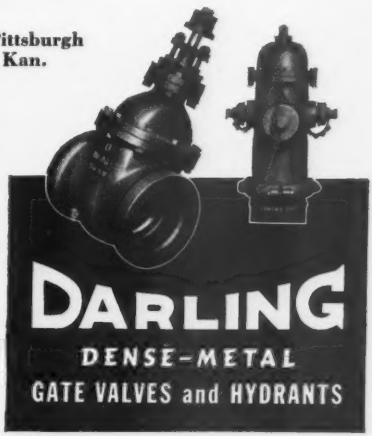

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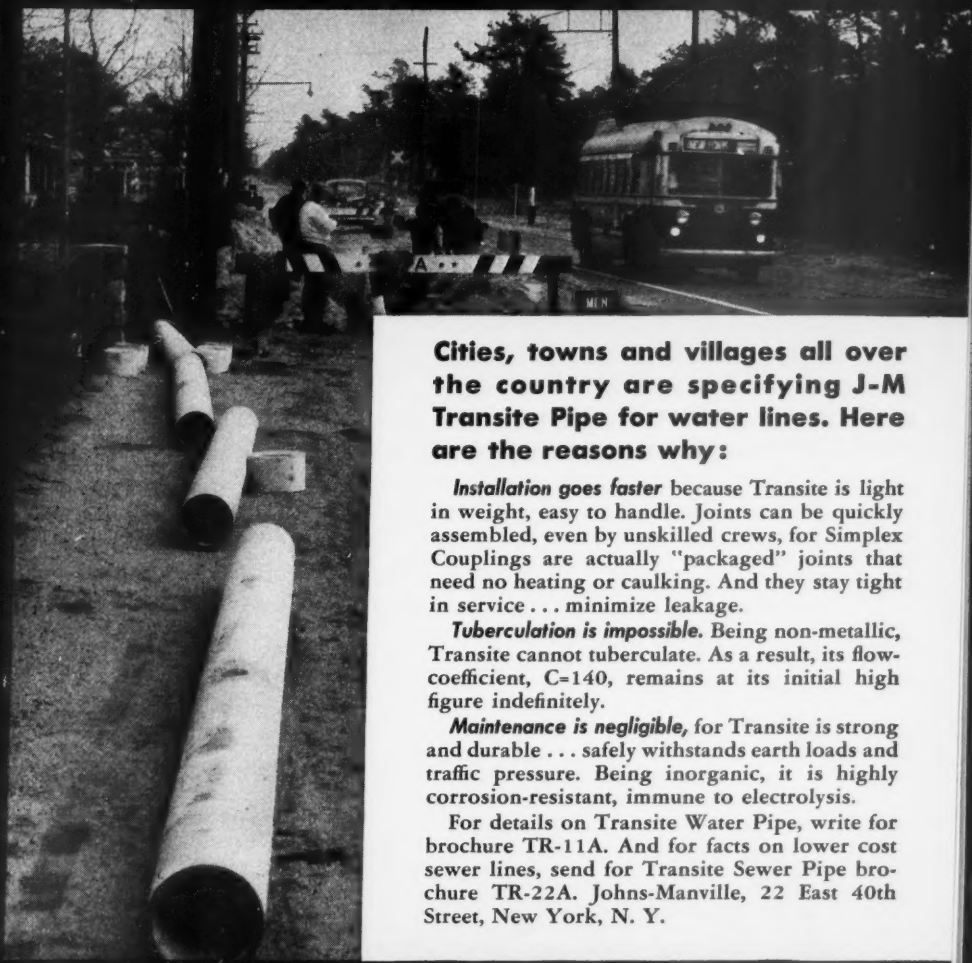

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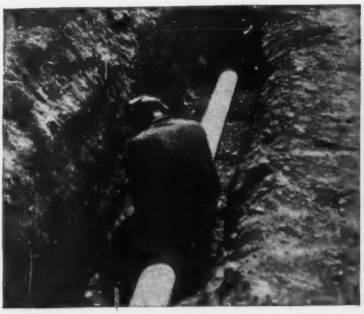

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And this is only one of the many advantages of the Kennedy SAFETOPS Their speed and economy of repair after accidental breakage, provided by the famous Safety Breakable Section, their ease of adjustment, their prompt response and dependable operation in fire service, their simple sturdy construction and attractive appearance — all these features have made Kennedy SAFETOPS the favored hydrant in progressive communities all over the country.

Write for full information on these unique hydrants and how they can be applied economically to your existing system, as well as on new extensions.

Bulletins sent on request

The Kennedy Valve Mfg. Co.

Elmira, N. Y.

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SAFETOP FIRE HYDRANT

REG. U.S. PAT. OFF.

EDDY

Valves and Hydrants

Distinctive Features of the EDDY Valves

ONLY three parts are moved by the Stem—the ball and the two gates. Each gate is hung from one of the trunnions on the ball. A convex surface at the base of each trunnion fits a concave surface on the back of the gate. This allows the gates to adjust themselves properly to their seats. . . . The gates, being free to revolve on the trunnions, do not always seat in the same position. Gates are center bearing and adjustable. They are forced to their seats with equal pressure at all points. There are two hooks on the ball on sizes 4-inch and larger, which loosely engage with the gates.

Distinctive Features of Our New Swivel Top Hydrants

THIS new hydrant has all the advantages of the popular EDDY fire hydrant, plus several additional features. . . . Nozzles are in a short flanged section of the standpipe, which may be unbolted and turned to different positions. . . . Should the standpipe be broken, only the casting below the swivel head need be replaced. . . . To raise the hydrant to conform to a new grading, simply insert a flanged extension piece below the swivelhead. No digging. To add a steamer nozzle at any time, it is only necessary to replace the swivel head with one having a steamer nozzle.

Besides valves and hydrants to meet the most exacting demands of waterworks service, the complete EDDY Line includes: valves for sewage disposal works; valves for steam, gas and oil; check valves, foot valves, plug valves, shear gates, indicator posts, etc. EDDY valves are made in three classes: Iron Body Bronze Mounted, All Bronze, and All Iron Valves.

Eddy Valve Co.

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Used by leading WATER WORKS for over 50 years. If proper weight is used, combined with good workmanship, your FINAL COST is assured, as REPAIRS are unnecessary. LEAD PIPE IS sufficiently ductile to conform to any sagging condition. There is no RUSTING of pipe or joints; no DISCOLORATION of water.

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Every atom pure lead

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RED LEAD

For rust-proofing tanks and pipes.

Write for Water Works specifications.

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DUTCH BOY WHITE LEAD

SOLDER

National-Boston Lead Co.

800 Albany Street **Boston, Mass.**

The FLO-WATCH is the meter that gives you the *facts*—at *low cost.*

IT WILL HELP YOU:

1. **Eliminate waste**
2. **Check pump operation and efficiency**
3. **Check boiler efficiency**
4. **Meter distribution lines**

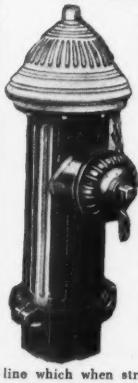
An investment in a Flo-watch Meter may easily pay an 100% dividend through savings effected.

Ask for Bulletin 318

BUILDERS·PROVIDENCE

IRON FOUNDRY RHODE ISLAND

THE A. P. SMITH MFG. CO.,
East Orange, New Jersey



Manufacturers of high-grade specialties in connection with water and gas department work, consisting of—
Machines for inserting valves under pressure, and for making large right angle connections to mains under pressure, and for inserting house service connections under pressure.
Also makers of—
Pipe Cutting Machines, each size being adjustable for use on three different sizes of pipe.
High-grade water works supplies, such as Hydrants, Valves, Meters, Corporation Cocks, etc.
We also make a new type of hydrant which has a breakable joint at the ground line which when struck with an impediment will break at a specified point and only a coupling has to be used to repair the hydrant, which will be back in service in twenty (20) minutes.

"HYDRANT WITH BREAKABLE COUPLING"

SMITH
WATER WORKS SPECIALTIES & SUPPLIES
THE A.P.SMITH MFG.CO. *East Orange,N.J*

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Ford double-lid meter box covers provide dead-air insulation in neck of cover and conserve heat in the meter box. Top lids are provided with the Ford Worm Lock. Write for catalog of meter box covers, double and single lid.

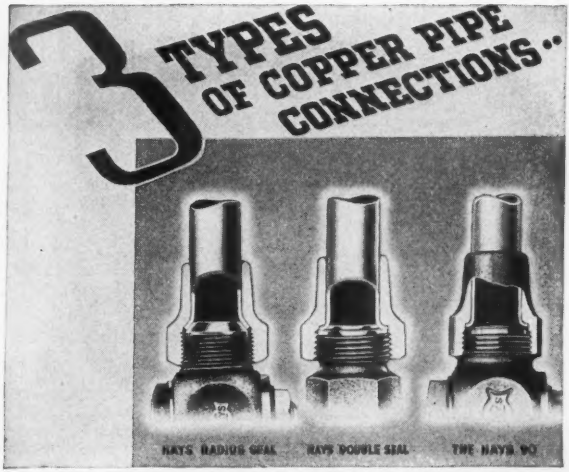
The Ford Meter Box Co.
WABASH, INDIANA

Sixtieth Annual Convention

New England Water Works Association

September 23 - 26, 1941

Hotel Statler
Boston, Massachusetts



Whatever tapping method or type of corporation cock your city uses, Hays can take off with copper pipe to the curb or to the meter. Hundreds of water works departments have found Hays permanent copper services readily adaptable to their needs at little or no extra cost over short life methods.

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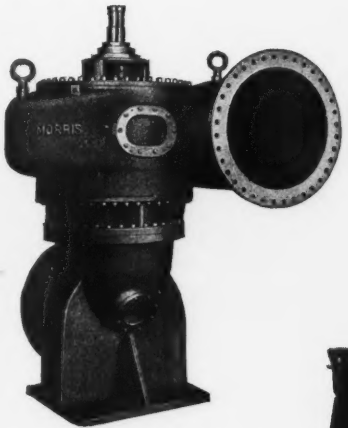
SINCE 1869

HAYS

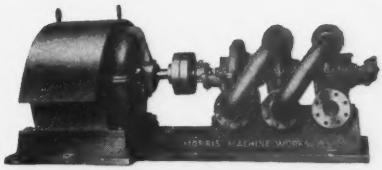
MORRIS

CENTRIFUGAL PUMPS

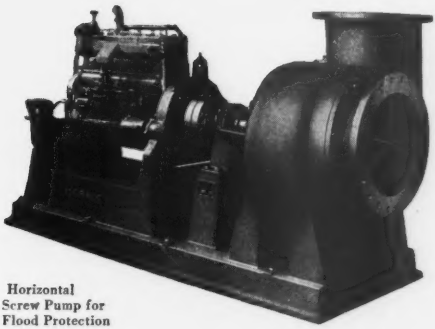
for every municipal service



Vertical Mixed Flow Pump
for Sewage



Three-Stage Horizontally-Split Double-Suction
Water Supply Pump



Horizontal
Screw Pump for
Flood Protection

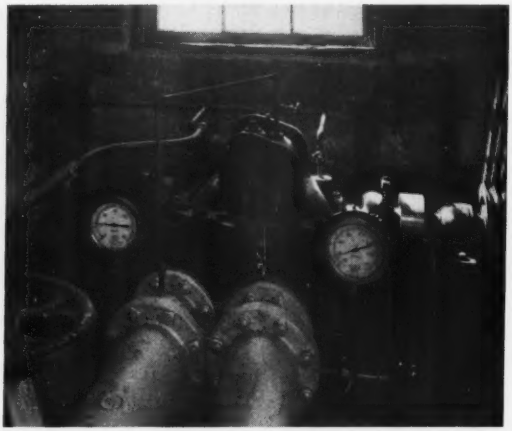
Vertical "Straightflo"
Pump for Drainage

Morris makes a centrifugal pump for every municipal duty, and engineers all over the country can testify to the uniformly high efficiency, trouble-free service, and low maintenance expense of Morris installations. Be sure to get a Morris quotation when you need a centrifugal pump for any duty.

STARKWEATHER ENGINEERING CO., INC.
NEWTON, MASSACHUSETTS

DE LAVAL

CENTRIFUGAL PUMPS FOR WATER WORKS



Two-stage Pump at West Groton, Mass., Water Works

The unit shown in the photograph is a De Laval two-stage Series type centrifugal pump which delivers water from driven wells into a stand-pipe at the rate of 500 GPM against a total head of 267 feet including about 14 feet suction lift.

Sand removal and priming is completely automatic so that the pump is primed at all times. The operation of the pump itself is also automatic by pressure control from the standpipe.

To avoid submergence of electrical apparatus by flooding such as occurred during the Hurricane of 1938, the driving motor, gasoline engine standby and all starting switchgear are placed at a higher elevation from which the pump is driven by V-belts.

We supplied all equipment and complete installation, under the direction of E. Worthington, Cons. Eng., Dedham, Mass., acting for the Water Board.

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DE LAVAL STEAM TURBINE COMPANY

TRENTON, NEW JERSEY

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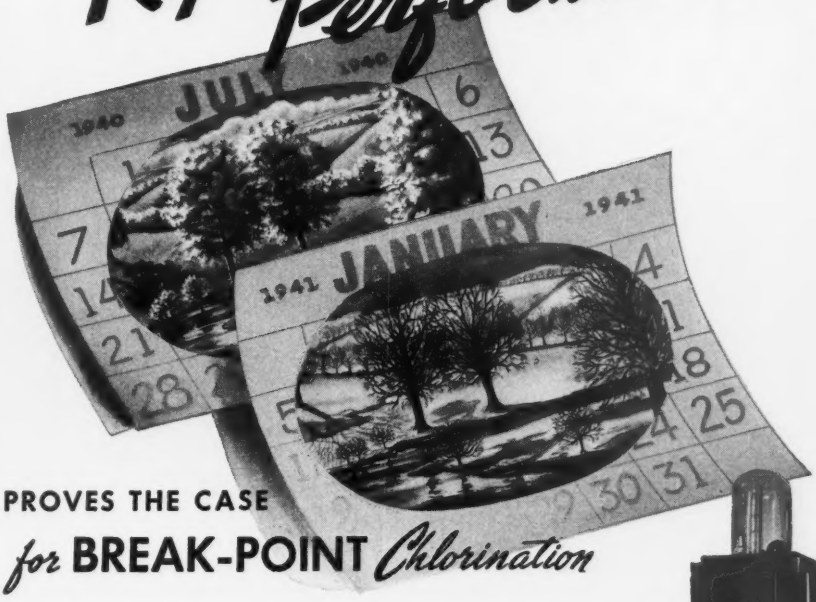

PROVES THE CASE
for BREAK-POINT *Chlorination*

Last July, in the face of a serious summer-time taste and odor situation, the town of Tarboro, N. C., applied Break-Point Chlorination to its water supply. It solved the problem. "I take off my hat to Break-Point . . ." said Bill Bryan, Superintendent of Tarboro's Water Purification Plant. Its use was continued until, with changing seasonal conditions, taste and odors were no longer a problem.

Then, suddenly, in the middle of January, "medicated tastes to extreme proportion developed in the Tar River." With equipment adequate for Break-Point Chlorination already on hand, the operation was shifted back to B-P a few hours after complaints first arose.

Again, Break-Point proved its case, producing an immediate improvement in taste.

After this successful *repeat performance*, Bill Bryan says, "I am going to keep Break-Point going from now on." Perhaps you, too, have a water supply in which taste and odor conditions may develop suddenly, without warning. If so, the preventive possibilities of year-round Break-Point Chlorination will be highly significant to you. A W&T representative will be glad to give you the facts and aid you in making the necessary tests . . . all without cost or obligation. Call him today, or write direct to Wallace & Tiernan, Newark, New Jersey.

SA-106

"The Only Safe Water is a Sterilized Water"

WALLACE & TIERNAN CO., Inc.

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For an Extremely Smooth Pipe Interior
SPECIFY

SPUN **BITUMASTIC** LINING
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This Will Assure You of
NO LOSS IN CAPACITY
NO TUBERCULATION OR INCRUSTATION
NO EFFECT ON WATER QUALITY

Recent Repeat Flow Test Shows High Williams Hazen Coefficient Unchanged After EIGHT YEARS

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CONE VALVES
(Patented Hydraulic Operating Device)

Used for
- ALTITUDE CONTROL
- PRESSURE RELIEF
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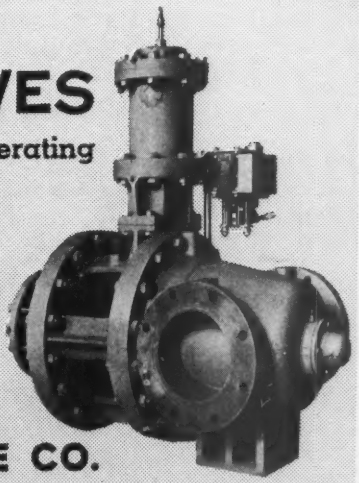

RENSSELAER VALVE CO.
TROY, NEW YORK

Make your plans to attend the 61st Annual Meeting of the American Water Works Association, Toronto, Ontario, Canada the week of June 22 to 26, 1941.

HOW COMPLETE
ARE YOUR REFERENCE FILES?

EXTENSIVE LITERATURE ON TURBINE, POWER, SPLIT CASE AND BUILTOGETHER CENTRIFUGAL PUMPS IS AVAILABLE TO YOU — TOGETHER WITH REMARKABLY DETAILED BULLETINS ON THE MOTORS AND DIESELS THAT DRIVE THESE PUMPS.

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Write to

FAIRBANKS, MORSE & CO.

178 Atlantic Avenue **Boston, Mass.**

Soon to be started: Installation of still another Fairbanks-Morse Diesel-driven Pump, this time for the Exeter, N. H., Water Company.

If you appreciate

SAVING

IN YOUR PIPE JOINTING

Using *Tegul*-MINERALEAD for your Bell & Spigot Main Jointing, you save in Time, Trouble, Labor . . . and Money too! ★ Quick sealing of initial leakage speeds the job ★ Skilled labor is not needed and storing *Tegul*-MINERALEAD outdoors you'll have no weather worries ★ It works with all diameters up to 48"—and larger if required ★ Joints are permanently tight and offer extraordinary resistance to thermal and mechanical shock ★ The first step to these savings and advantages is to write for more information ★ Address The ATLAS MINERAL Products Company of Pa., Mertztown, Pennsylvania.

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THE INGOT FORM JOINTING COMPOUND FOR BELL AND SPIGOT MAINS

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MANUFACTURERS

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Special Castings for Water and Gas

Also Flange Pipe and Fittings

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New York Office: E. A. NOONAN, 225 Broadway

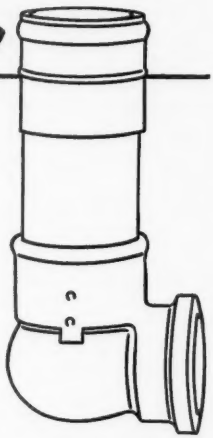
70 years ago the removable barrel was a *basic* improvement in hydrant design. Today it is still the *basic* advantage of the Mathews Hydrant. Here's why:

❶ A smashed barrel, unscrewed and replaced with a spare barrel, makes an immediate repair without digging.

❷ Inserting a spare barrel allows inspection or overhaul in the shop of all operating parts, including main and drain valve *seats*.

❸ Without digging, modern steamer nozzles can be added, and grade levels can be changed up or *down*.

❹ The barrel, surrounded by its loose protection case, gives double impact strength and prevents frost-heaved ground from straining the hydrant.



SO—if you are buying new hydrants, specify Mathews. If you *have* Mathews, check your supply of spare barrels this week.

MATHEWS HYDRANTS

Made by R. D. WOOD COMPANY

Manufacturers of Sand Spun Pipe (centrifugally cast in sand molds) and R. D. Wood heavy-duty gate valves for water works

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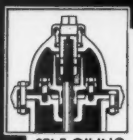
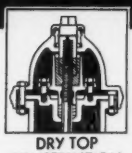
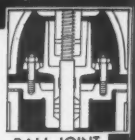

SELF-OILING TOP


DRY TOP CONSTRUCTION

ALL WEATHER CAP

NON-FREEZING OIL


BALL JOINT CONSTRUCTION

HYDRANT EASILY EXTENDED

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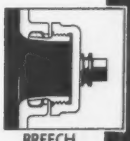
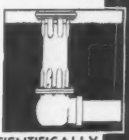

BREECH LOCKED NOZZLES

REPLACEABLE SAFETY-FLANGE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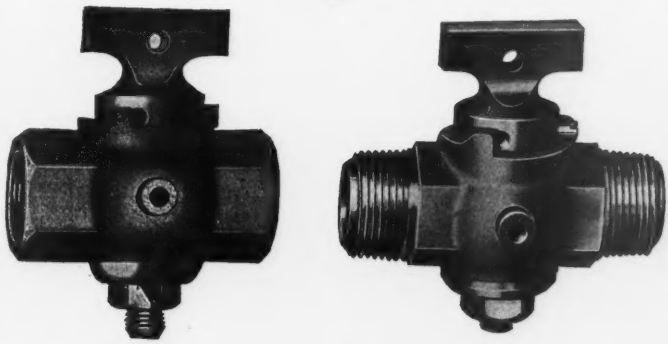

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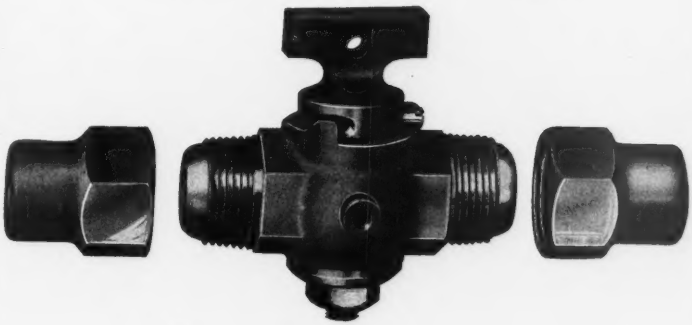

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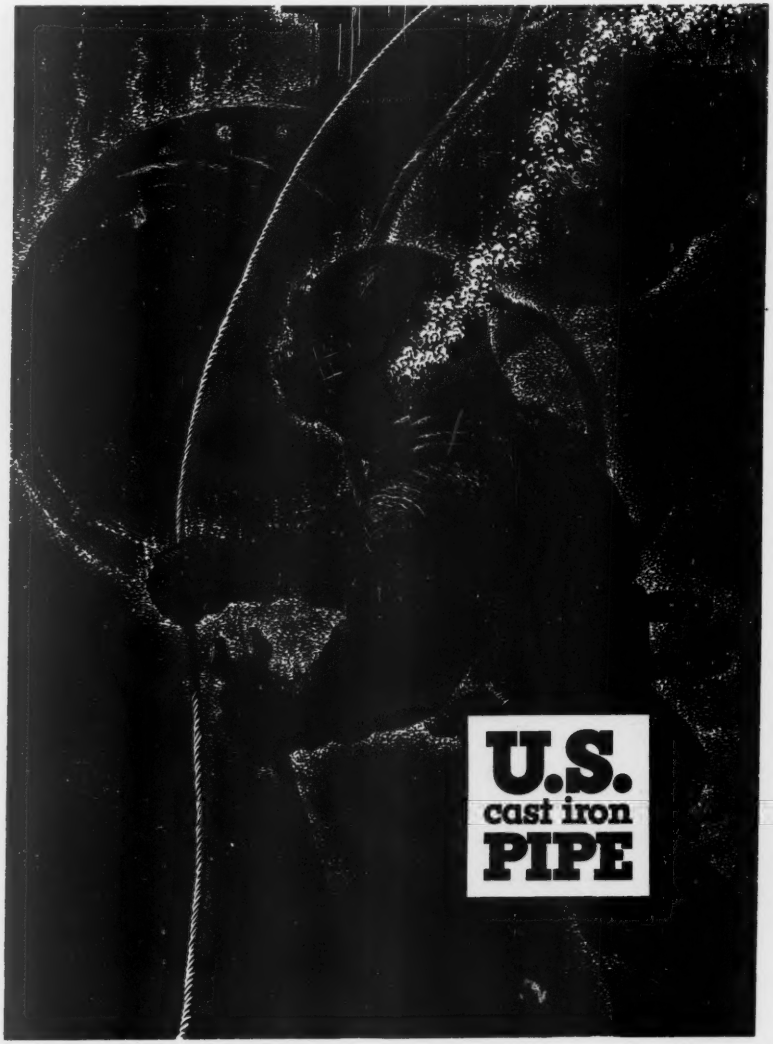

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